DESIGN MANUAL FOR SEWERS AND WATERMAINS

Sponsored by

the Municipal Engineers Association

and the

Ministry of the Environment

Ontario

INDEX

PROJECT PROCEDURES

by

Mr. T. A. McMullen, P. Eng.

PROJECT PROCEDURES

This topic may seem to be an unusual way to open a course on Sewer Design; but hopefully, after I have finished my presentation, you will see why it has been included, and the important relationship between what you are doing (i.e., designing the project), and what others are doing concurrently (i.e., processing the project through the required channels).

Procedures are frequently referred to as "Red Tape", and indeed this is what it seems like. I personally find red tape very frustrating, but sometimes when you analyze procedures step by step, it becomes evident that it is all part of the democratic procedure.

It must be recognized that as individuals we are not likely to bring about any major procedural changes; as many of them are laid down in Provincial Acts such as the Local Improvement Act, and Expropriation Procedures Act, etc. Hence, our job should be to understand the procedures as well as possible, and by understanding them we can guide the project through the maze in the most efficient manner, and with a minimum of time loss.

The degree to which you will be exposed to the procedural steps depends to a large extent on the size of the Municipality you work for, and whether you are Municipal staff or a Consultant working for a Municipality. I should like to point out that working for a Consultant does not mean that you are not involved with project procedures. Actually, the procedures can be even more complex for a Consultant; as he is working with different Municipalities, each one having its own method of handling the project with respect to approvals. In Scarborough we frequently have Consultants propose schedules for design and construction which are most unreasonable, simply because they lack the knowledge on the multitude of steps a project must follow prior to actual construction.

In the large Metro areas, frequently there are administrative sections of which one of their functions is to carry out certain of the procedural steps in local improvements and capital works. In smaller Municipalities this work may be handled by the Clerk himself in conjunction with the Engineer; but regardless, it usually seems that the Engineer acts as the catalyst in directing the project through the appropriate stages.

It seems that everything in life is constantly becoming more complicated, and nowhere is this more true than in the field of government

projects. Gone are the days when a Municipal Engineer could decide what should be done in terms of improvements in the municipality such as road widenings, sewers etc., and obtain Council approval, then proceed with the work with minimal interference by the public. The days of participatory democracy and public involvement are now with us. We are now dealing with an enlightened public. Unfortunately, public involvement is more often than not negative; or in the form of opposition to the project. Rarely is there much involvement from those in favour of a project. Senior levels of government are putting more power into the hands of the people. If we need an easement for a sewer, no longer is it a simple matter to expropriate the land and count on possession in perhaps one month. The Expropriations Act has been changed considerably, now providing for public hearings as to the necessity, and fairly long proceedings such that it can now take 9 to 10 months or more to expropriate an easement.

This is only one example; but what all this points out is that since public opposition is so prevalent on our work today and the individual has so many more rights, we must proceed very carefully with our plans and designs, and in strict accordance with all Acts or Procedures which are laid down. A mistake discovered mid-way through an expropriation proceeding can mean that the whole procedure must be started over. This is not only embarrassing to the Municipality, but time-consuming, and it can be very costly.

Obviously in the space of one hour it is not possible for me to cover all aspects of the various Municipal procedures. They will of course vary significantly from area to area, and project to project; and for this reason it is not my intent to get too detailed or specific. Anything presented in this paper should be considered of a general informative nature, and is not intended for specific uses.

I will attempt to give a general insight into the methods of financing municipal projects, outline some of the more commonly used Local Improvement Act procedures, and the Expropriation Act procedures.

On the flow chart which I will distribute at the course, I will attempt to show schematically how these various procedures fit into the flow of a project, and to show how they can influence the timing of construction.

Procedures could arbitrarily be broken down into three categories, as follows :-

1. Design approvals.
2. Financial approvals
3. Miscellaneous procedures.

Under the heading of "Design Approval" I would include such things as Ministry of the Environment (formerly Ontario Water Resources Commission) approval, Ministry of Transportation & Communications approval, and in the Metro area on sewer projects, Metro Works Department approval is required. Special projects will frequently require design approval from other Authorities, depending on the area you are working in, and the nature of the project; but I can only speak from experience in the Metro Toronto area.

Generally speaking, design approvals are relatively straightforward to obtain, and do not involve long periods of time. Our experience in Scarborough is that all design approvals can normally be obtained within 3 - 4 weeks after applying.

Financial approvals can be considerably more time-consuming than the design approvals. The method of financing chosen for the project has much to do with the required procedures. Any projects which are financed from Current Funds or Current Reserve Funds normally only need Council approval of the expenditure, together with the necessary design approvals, before getting underway.

However, most large scale projects are financed as Capital Works. Under the general heading of 'Capital Works' there are two main methods of processing projects, both of which involve the sale of debentures to raise the money for the project, require O.M.B. approval, and again in the Metro Toronto area approval to debenture a project is required from the Metropolitan Corporation. These methods are as follows:

1. Local Improvements: Such projects as sidewalks, curbs, local sewers and watermains, and house connections, are frequently undertaken as local improvements and a portion or all of the cost is directly assessed against the benefitting properties. Generally the sum of the Corporation and Owner's share (i.e., the total cost of the work less the subsidy) is financed through the sale of debentures. To undertake a project as a local improvement requires that all procedures outlined in the appropriate section of the Local Improvement Act be followed. Later in this article I outline briefly some of the procedures required by Sections 12 and 8 of the Local Improvement Act.

2. Capital Projects are different from Local Improvement projects in that none of the cost of the work is directly assessed to abutting owners.

Typical projects to be financed this way are trunk storm and sanitary sewers, watercourse improvements, and major roadways and trunk watermains. The money for the work is raised through the sale of debentures with annual repayments over the debenture term from the general tax rate.

An indication of the timing required to obtain the various financial approvals is shown on the flow chart.

Under the heading of "Miscellaneous Procedures" I would include such things as expropriation of land, acquisition of railway pipe crossing agreements, and Canadian Transport Commission approval of road crossing of railway tracks, and numerous others which will be peculiar to a particular job. Again later in this article I have outlined some of the procedures to be followed under the new Expropriation Act, and on the flow chart have shown indications of time requirements for some of the miscellaneous procedures.

LOCAL IMPROVEMENT ACT:

Section 12 Procedures: (for use when Council is proceeding under the Initiative Plan).

- List all the projects which are approved by Council for construction in a particular year and to be financed through the sale of debentures. Normally the projects are grouped in the list according to type (i.e., watermains, sewers) and method of payment (i.e., Local Improvement or Capital project etc.). This step in itself is not part of the Section 12 Procedures but is done with a view to obtaining Bulk Approval of the budget from Ontario Municipal Board.
- O.M.B. Bulk Approval of this list is obtained.
- An Engineer's Report is prepared for each local improvement project on the list. (See typical examples attached).
- On each project letters are sent to all the effected owners, advising them of the work and giving them 30 days in which to object, if they wish.
- If less than a majority of owners representing at least half the value of the lots liable to be assessed object, then after passing a Construction By-Law, the project construction can proceed.

- If sufficient objections are received to hold up the project, the Clerk must report to Council, and Council decides whether to drop the project or proceed under Section 8 of the Local Improvement Act.

SECTION 8 PROCEDURES:

If Section 8 is selected originally, or if it is decided to use Section 8 after having been turned down using Section 12, the following procedures are to be adhered to;

- Council must approve the project as a Local Improvement by a vote of 2/3 of all Members.
- It is necessary to publish notice of intention to apply for O.M.B. approval of the project in the local papers, once a week for two weeks.
- Any owner has 21 days from the date of the first notice to file an objection.
- If there are no objections, O.M.B. approval is normally received and it is then necessary to pass a Construction By-law.
- If there are objections, there will normally be an O.M.B. Hearing. It can take a few months to obtain a Hearing date, and the Municipality must prepare and present a formal presentation at the Hearing. If after the Hearing the O.M.B. approval of the project is received, a Construction By-law is passed and the construction can proceed.

EXPROPRIATION PROCEDURES:

Prior to the passing of the New Expropriation Act, all that was necessary for a municipality to obtain land or easements was for Council to authorize the land acquisition, survey the property, and prepare and file an expropriation plan, and take possession; the procedure could be carried out in as little time as one month.

Under the new Act, which is now in effect, the following is an outline of the procedures which must be followed to acquire easements or ownership.

1. Notify all owners in writing of the Municipality's intent to expropriate.

2. Concurrent with 1 above, an ad is placed in the local newspapers and is published weekly for 3 consecutive weeks. This ad advises of the Municipality's intent to expropriate.

3. The owners are given 30 days in which to file an objection and request a Hearing.

4. If a Hearing is requested, it must be arranged, and it can take 5 or 6 weeks to get a Hearing. The purpose of the Hearing is to determine whether the taking of the property by the expropriating authority is sound, just, and reasonably necessary.

5. The Municipality must prepare grounds upon which it intends to rely at the Hearing (i.e., why the land is being taken). This requires preparation of plans, documentation of evidence, and other rather extensive preparations, all with a view to providing proof at the Hearing that the taking of the land is reasonably necessary.

6. The Hearing is held, and the Inquiry Officer who runs the Hearing prepares a report after the Hearing,with his recommendations. This is forwarded to Council and can take 2 or 3 weeks to obtain.

7. The owners, or all parties at the Inquiry, are advised of Council's recommendation with respect to the expropriation. Up to 90 days are allowed for this purpose, but it normally is done immediately.

8. The Expropriation By-law is passed.

9. The Expropriation Plan is filed in the Registry Office.

10. Within 30 days of registering the Expropriation Plan, all the owners affected are sent a Notice of Expropriation, Notice of Election as to date when owner wishes to fix compensation, and a copy of the Expropriation Plan.

11. 90 days after the mailing of the information referred to in 10 above, possession is obtained.

12. An offer of compensation must be made in the 90 day interval referred to in 11 above.

At this stage of the procedures we have possession of the land, which is our major consideration as far as construction planning is concerned. It is interesting to note, however, that there are still further steps to be taken by those responsible for acquiring the land.

It is very important to note with respect to all the foregoing, that if at any point in the above procedures it is decided that ..say.. a little more land is required .. then it is necessary to start over on the entire procedure, thereby delaying the project several months. Hence, it is most important when initially establishing land requirements to ensure that they are correct.

The total time elapsed for an expropriation procedure is a minimum of approximately 6 months - if no inquiry is required. If an inquiry is required, the total time will be 9 months minimum.

BOROUGH OF SCARBOROUGH

FILE NO...............

Works Department

ENGINEER'S REPORT ON LOCAL IMPROVEMENT

DATE..................

To the Mayor and Council,
Borough of Scarborough.

I recommend that the following work be constructed as a Local Improvement under the provisions of the Local Improvement Act, the said work being, in my opinion, necessary.

Method of Procedure-- Section 12, Local Improvement Act

Nature of Improvement-- Sidewalk

Street Danzig Street (both sides)

From (N/S) Kitchener Rd. to E.L. #30 Danzig St.
(S/S) Poplar Rd. to W.L. #27 Danzig St.

XXX

Length of Work 720 ft.

The real property which will be immediately benefitted is the property fronting or abutting the portion of the street named above and is the land to be assessed.

The probable cost of the work is $3,960.00 and the estimated lifetime is 11 years. The debentures, bearing interest at 7 per cent, are to extend over a period of 10 years. The proportion of the cost to be assessed against the property benefitted is $ 847.00 and the remaining $ 3,113.00 which is made up of intersection, flankage and 50% of owners normal share of the cost

is to be borne by the Borough.

The proportions in which the said assessment should be made on the various portions of the lands benefitted as aforesaid is an equal rate per foot according to frontage. The assessable frontage is 308 feet, and the non-assessable frontage is 412 feet which is made up of intersection and flankage.

The probable cost of the work per foot is $ 5.50 and the probable cost per foot of frontage is $ 2.75 and the annual special rate is .39xx 15 per foot of frontage.

18-06-031A

COMMISSIONER OF WORKS.

BOROUGH OF SCARBOROUGH

FILE NO...............

Works Department

ENGINEER'S REPORT ON LOCAL IMPROVEMENT

DATE..................

To the Mayor and Council,
Borough of Scarborough.

I recommend that the following work be constructed as a Local Improvement under the provisions of the Local Improvement Act, the said work being, in my opinion, necessary.

Method of Procedure-- Section 12, Local Improvement Act

Nature of Improvement--Storm & Sanitary Sewers

Street Marilyn Avenue

From Kennedy Road

To Approx. 840 ft. easterly

Length of Work 855 ft.

The real property which will be immediately benefitted is the property fronting or abutting the portion of the street named above and is the land to be assessed.

The probable cost of the work is $60,700.00 and the estimated lifetime is 16 years. The debentures, bearing interest at 7¼ per cent, are to extend over a period of 15 years. The proportion of the cost to be assessed against the property benefitted is $14,600.00 and the remaining $ 46,100.00 which is made up of intersection, flankage and oversize is to be borne by the Borough. Of the Borough share of $46,100.00, $14,700.00 is expected to be recovered by M.T.C. subsidy

The proportions in which the said assessment should be made on the various portions of the lands benefitted as aforesaid is an equal rate per foot according to frontage. The assessable frontage is 1460 feet, and the non-assessable frontage is 280 feet which is made up of intersection and flankage.

The probable **cost** of the work per foot is $ 70.99 and the probable cost per foot of **frontage** is $10.00 and the annual special rate is 1.1153 per foot of **frontage**.

18-06-031A

COMMISSIONER OF WORKS.

ELEMENTS OF GOOD DRAWINGS

INTRODUCTION

The value of an engineering drawing lies in the ability of the designer to convey his thoughts in a graphic language to others with limited time who must have the necessary information to construct the service, structure or machine envisaged by the designer.

There are many users of the engineering drawing. The estimator must use the drawing for preparation of costs for budgeting or tendering purposes. The lack of information or incorrect information on a drawing can lead to embarrassing situations for those who must make financial arrangements for the project. Extra costs for items not shown can be quite substantial and exceed budget limitations at times. Tenders can be unbalanced by errors on the drawing. If an error of ½" in thickness of asphaltic concrete surfacing were made on a drawing, the cost difference would be over $5,000 on one mile of a 50-foot wide paved road or $1 per foot.

The surveyor, who must do the construction layout, requires accuracy of dimensions in order to ensure that every line and grade is properly set. Many errors in drawings reveal themselves at this stage which can lead to costly confusion.

The contractor who must construct the work requires total information so that he may order the right type and quantity of materials, supply the proper machinery and employ the required number of men.

The project engineer and inspector who oversee the total performance of the contractor, compile reports for progress of work and for payments must have complete drawings and cannot ask for items to be constructed that are not shown unless costly extras are allowed.

In summary then, the drawing should be accurate in dimensions and spelling, clear in concept, easily understood, neat in layout with good

ELEMENTS OF GOOD DRAWINGS

AND

FIELD INFORMATION

by

Mr. W. A. Wright, P. Eng.

TEMP. NO.	Standard Pen Nos.											
			Reservoir Pen Nos.									
	0000	000	00	0	1	2	3	4	5	6	7	8
50	A*											
60	A*	A	B	C								
80	A	A*	B	C								
100	A	A	B*	C	D							
120		A	B	C*	D	E						
140		A	B	C	D*	E	F					
175		A	B	C	D	E*	F	G				
200		A	B	C	D	E	F*	G	H			
240		A	B	C	D	E	F*	G	H			
290			B	C	D	E	F	G*	H	K		
350			B	C	D	E	F	G	H	K		
425			B	C	D	E	F	G	H	K	L	
500			B	C	D	E	F	G	H	K	L	M

FIGURE NO. I

penmanship and complete in all respects.

THE ELEMENTS

Since the persons attending this course are designers and engineers with considerable experience, not too much time will be spent on lines and lettering which are the basic elements of drawing. However, perhaps we could touch on a few of the highlights of lines and lettering for a moment.

LETTERING

Lettering should be consistent in size for similar items. Items of greater importance can be emphasized with bold large-sized letters. Several different sizes and weights of lettering could be used on one drawing but all should be selected with due consideration for the size and purpose of the plan to be drawn. There is an unlimited variety of sizes and styles of letters available with much work being done mechanically but all should be clear and legible.

SLIDE NO. I (FIGURE NO. I)

This slide shows template sizes, pen numbers and indicates by asteriskes the recommended pen for the particular letter although other pen sizes could be used. The entire alphabet is not represented but it might be well to mention that each letter has a defined shape. For example, the horizontal bar on A is below centre, whereas for E and F, the bar is slightly above centre. It may seem elementary but each draftsman should memorize the proper shape and proportion of each letter to achieve good balance for freehand lettering.

LINES

In general, lighter lines are used for dimension lines, centre lines and existing conditions. Medium weight lines are used for such things as street lines, building outlines and heavy lines for proposed

works on contract drawings. Some offices have a table of standard weights of lines to be used by all draftsmen for uniformity and clarity. It would be impractical for me to suggest a particular line weight for every case but the end product should have a well-balanced appearance. Certain types of dotted, dashed or a combination of both dots and dashes are commonly used to represent specific items such as Bell Canada cables, Consumers' Gas lines, sewers, watermains, etc., and vary from one municipality to the next. The Metropolitan Toronto Public Utilities Co-ordinating Committee has issued a Technical Legend in an attempt to standardize these within the Metropolitan area. A copy may be obtained from the Municipality of Metropolitan Toronto Roads Department.

LAYOUT (SLIDE NO. 2) (FIGURE NO. 2)

Our experience indicates that many draftsmen have on-the-job training only and lack formal academic training in such things as orthographic projection and pictorial representation. This slide is intended to represent a poor drawing illustrating some of the most common errors that occur. For example:

1. Layout - views in the wrong angles.
2. Dimensional errors
3. Arrowheads
4. Tangent points to circles
5. Cross-overs at corners
6. Smudges
7. Inconsistent letters - capital & lower case in the same word
8. Spelling errors - orthographic
9. Notes randomly placed
10. Insufficient information to construct the object
11. Poor spacing of lettering
12. Radius wrongly shown

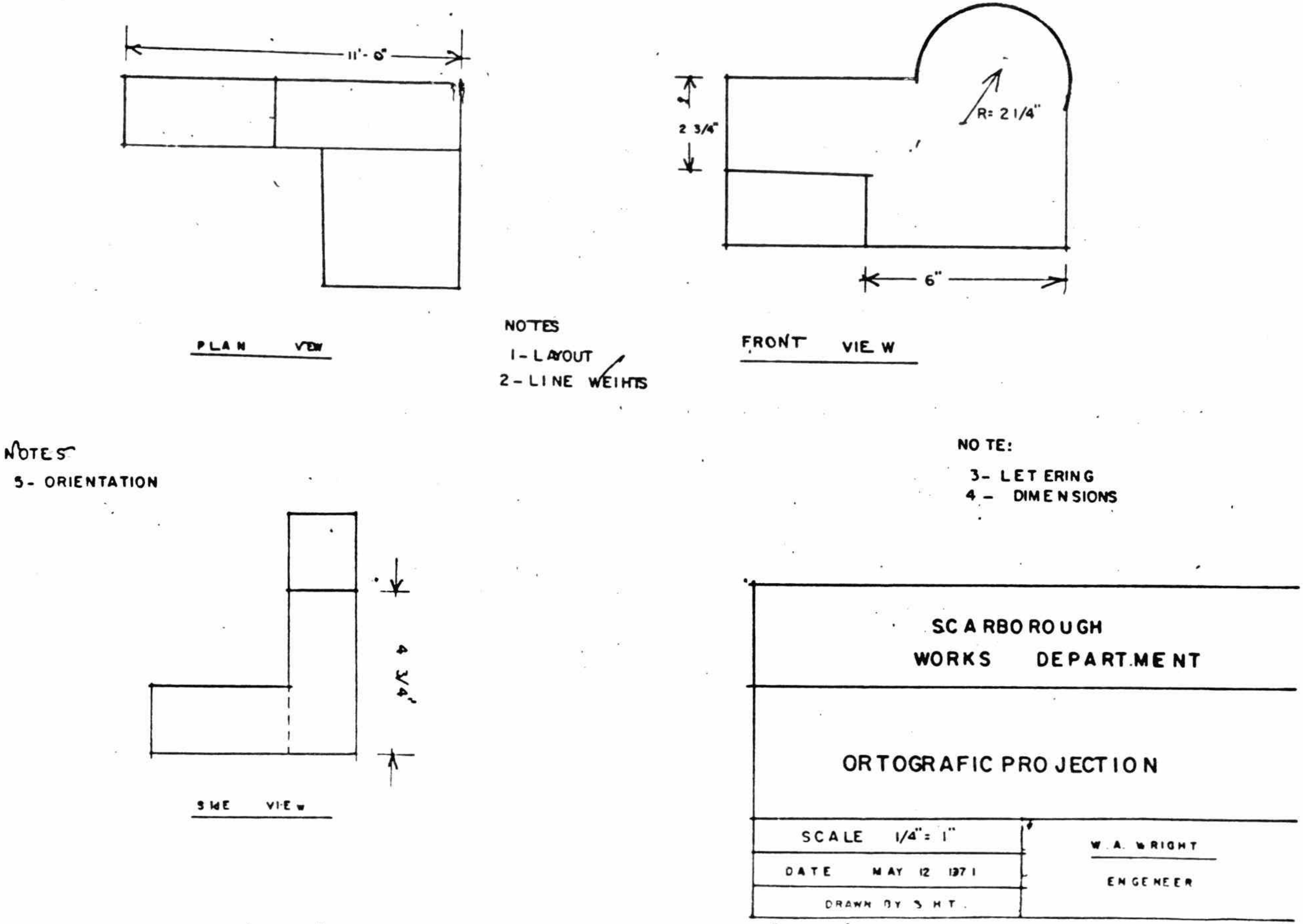

11'- 0"
PLAN VIEW
2 3/4"
R= 2 1/4"
6"
FRONT VIEW
NOTES
1 - LAYOUT
2 - LINE WEIHTS
NOTES
5 - ORIENTATION
NOTE:
3 - LETERING
4 - DIMENSIONS
4 3/4"
SIDE VIEW
SCARBOROUGH
WORKS DEPARTMENT
ORTOGRAFIC PROJECTION
SCALE 1/4" = 1"
DATE MAY 12 1971
DRAWN BY S.H.T.
W. A. WRIGHT
ENGENEER

FIGURE NO. 2

ORTHOGRAPHIC PROJECTION (SLIDE NO. 3) (FIGURE NO. 3)

The same object is shown here properly drawn:

1. Correct orthographic projection
2. Correct dimensions
3. Proper arrowheads
4. Tangent point to circle correct
5. Corners not over-extended
6. Clear - no smudges
7. Good selection of letter sizes
8. Correct spelling
9. Notes well-placed
10. Complete information to construct
11. Consistent letter spacing
12. Radius correct.

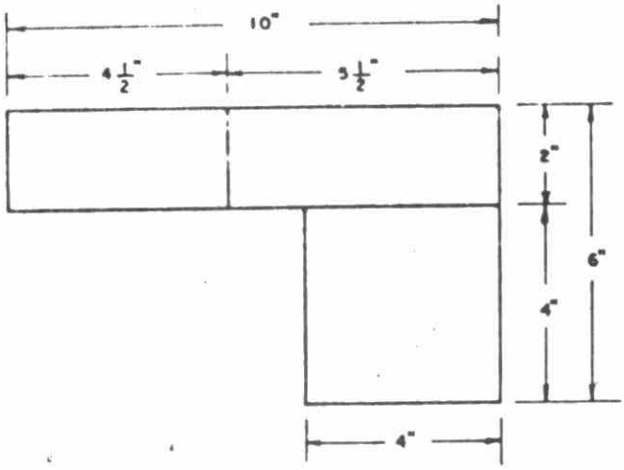
The notes in the lower part of the drawing were made with reference to the contents of the Title Block.

"Where" refers to the location of the work to be constructed. On a municipal engineering drawing this would be placed where "Orthographic Projection" is noted on this drawing and identifies the site of the works to be carried out.

"What" in event that it is an object such as a manhole, catch basin, etc., without a particular designated location would be placed in the main part of the Title Block to identify the particular object.

"Who" refers to those persons involved in the production, design and approval of the drawing. If questions regarding the intent of the drawing arise, it is an easy matter to trace the parties responsible from the signatures in the Title Block. Usually the draftsman, or designer, plans examiner and engineer will be required to sign a drawing. The status of drawings without signatures is questionable and can cause problems.

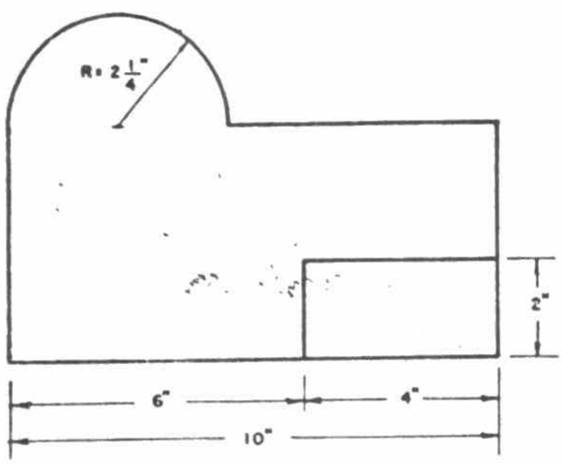
"When" refers, of course, to the date of production of the drawing and is an important item to be included. Dates become especially important on drawings when the drawings are incorporated as a part of contracts or subdivision agreements. After execution of such documents,



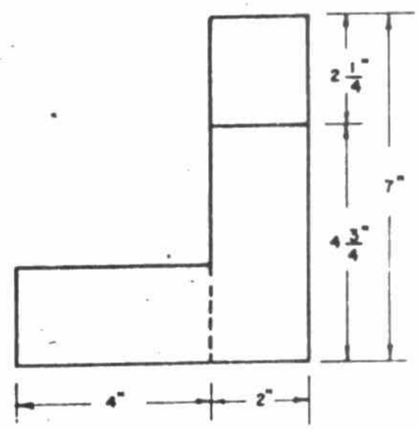
PLAN VIEW

NOTES

1-LAYOUT - ORTHOGRAPHIC
2-LINE WEIGHTS.
3-LETTERING.
4-DIMENSIONS.
5-ORIENTATION.



FRONT VIEW



SIDE VIEW

NOTES

1-WHERE
2-WHAT
3-WHO
4-WHEN
5-DRAWING Nº.
6-REVISIONS

1-	
Nº	REVISIONS
SCARBOROUGH WORKS DEPARTMENT	
ORTHOGRAPHIC PROJECTION	
SCALE 1/4" = 1"	W. A. WRIGHT ENGINEER
DATE MAY 12, 1971	
DRAWN BY C. AOWN	DWG. No. 1
CHECKED BY N. ORPIN	

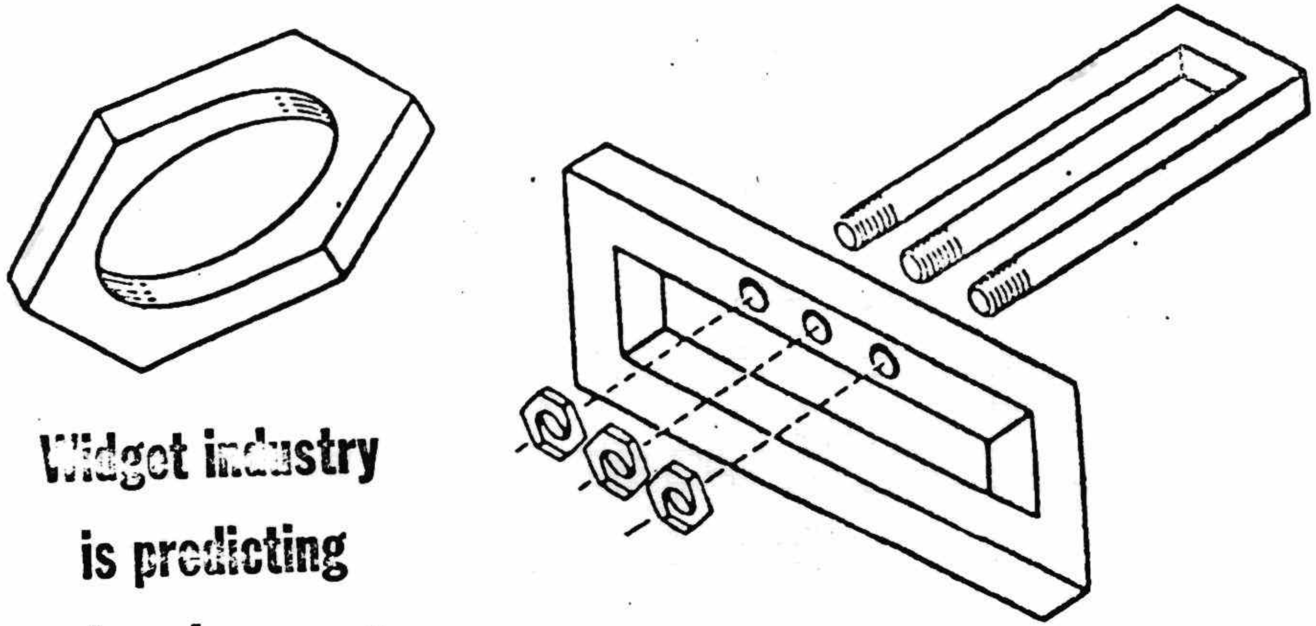
FIGURE NO. 3

revisions to any drawings which form a part of the contract or agreement are invalid unless legalized by both parties to the contract or agreement.

The "Drawing Number" is, of course, necessary and important for ready reference, filing, etc. There are many systems used for numbering of drawings, some of which use a letter to designate a standard paper size. Some use the year (i.e. "71"), others use letters and numbers for many purposes of identification such as "S" for sewers and "R" for roads. The systems and their merits are too numerous to mention here.

"Revisions" to drawings should always be noted and dated and are usually placed in a column provided above the Title Block. What appears to be a minor revision to a draftsman by the change of a dimension or line can be of major consequence to the users of the drawing. For instance, we recently altered the width of Scarborough sidewalks from 4'6" to 5'0" and when we consider that 10 to 15 miles of sidewalks are constructed annually in Scarborough, a considerable amount of money is involved due to this change. We compensated to a large degree for this additional width by decreasing the depth of walk required at most driveways.

This is a relatively simple object to draw - complicated objects are usually a combination of many simple objects. In order to convey the complete picture to others, I believe it is essential for draftsmen to familiarize themselves with orthographic projection. Incidentally, there are six possible views - plan (top), left side, right side, front, rear and bottom. Seldom are all six views used. An excellent reference text for increasing your knowledge of formal drafting practices is "Engineering Drawing" by Thomas French and Charles Vierck published by McGraw Hill Company. This reference text covers everything in drafting including Instruments and Their Use, Applied Geometry, Othographic



Widget industry is predicting a hopeless year

A growing credibility gap between designers and manufacturers likely will result in another year of zero production for the widget industry.

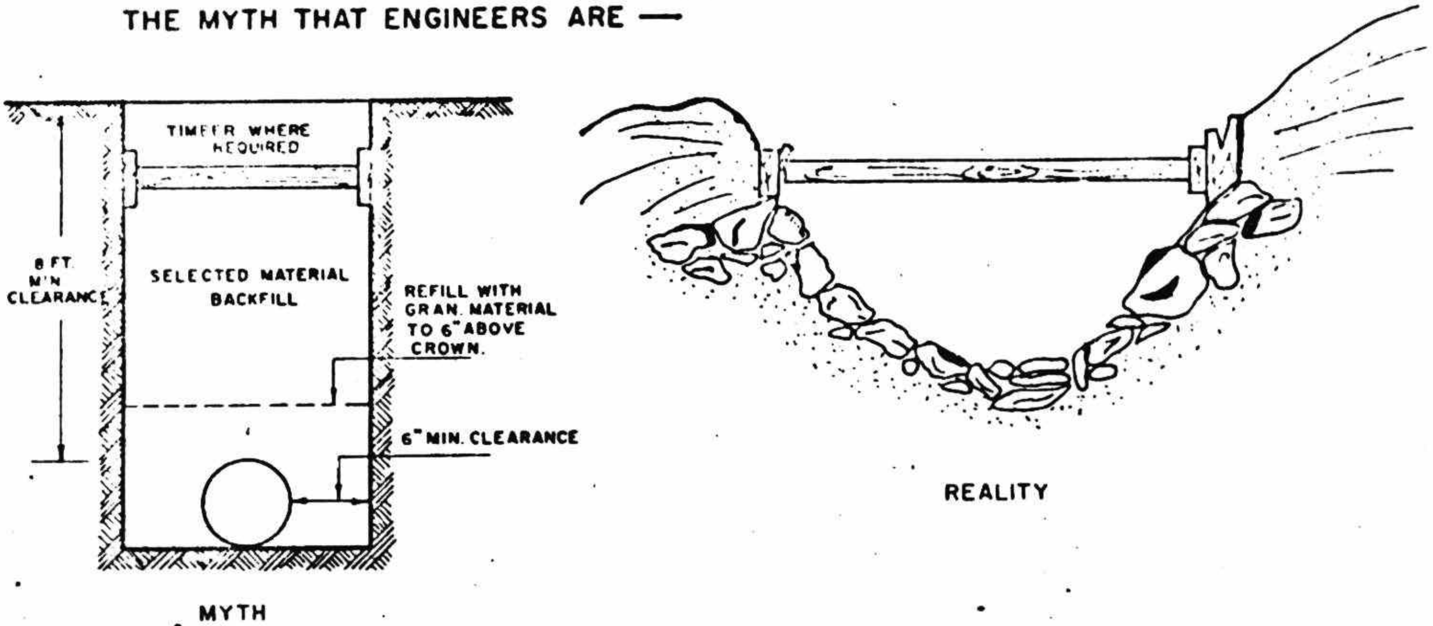
The designers claim that their latest model, featuring the newly invented eccentric triprong (above right), represents the ultimate in elegant complexity, and they can't understand the production delay.

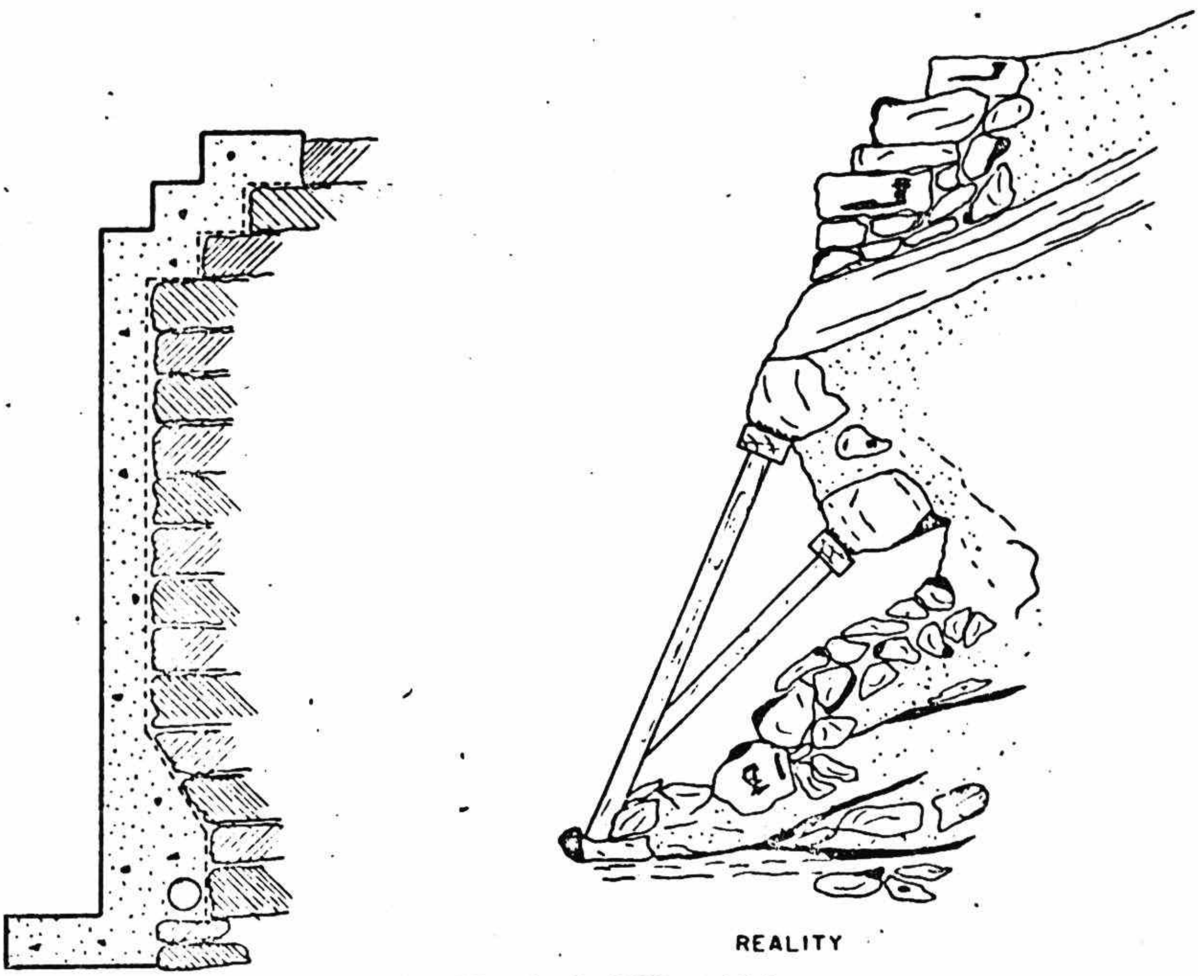
But plant managers say that while the ends of the new device are simple enough to make, they still are having trouble with the middle sections.

Marketing men report that the ambihelical hexnut (above left), once considered a major breakthrough, is not living up to expectations.

The internal wrangling over design is compounded by the fact that no one has yet figured out a use for widgets, which, of course, makes them difficult to sell. Unless these problems can be sorted out soon, experts fear that the entire widget industry will degenerate into utter chaos.

THE MYTH THAT ENGINEERS ARE —





—PRACTICAL FELLOWS DIES HARD.

MYTH

DRAFTMAN'S DELUSIONS

FIGURE NO. 5

Projection, Pictorial Drawing and Sketching, etc., and is well worth having available in a drawing office not only for the novice draftsman but also as a ready reference to the experienced draftsmen for some things they may have long forgotten.

PRACTICAL DRAWINGS (SLIDE OF WIDGET NO. 4) (FIGURE NO. 4)

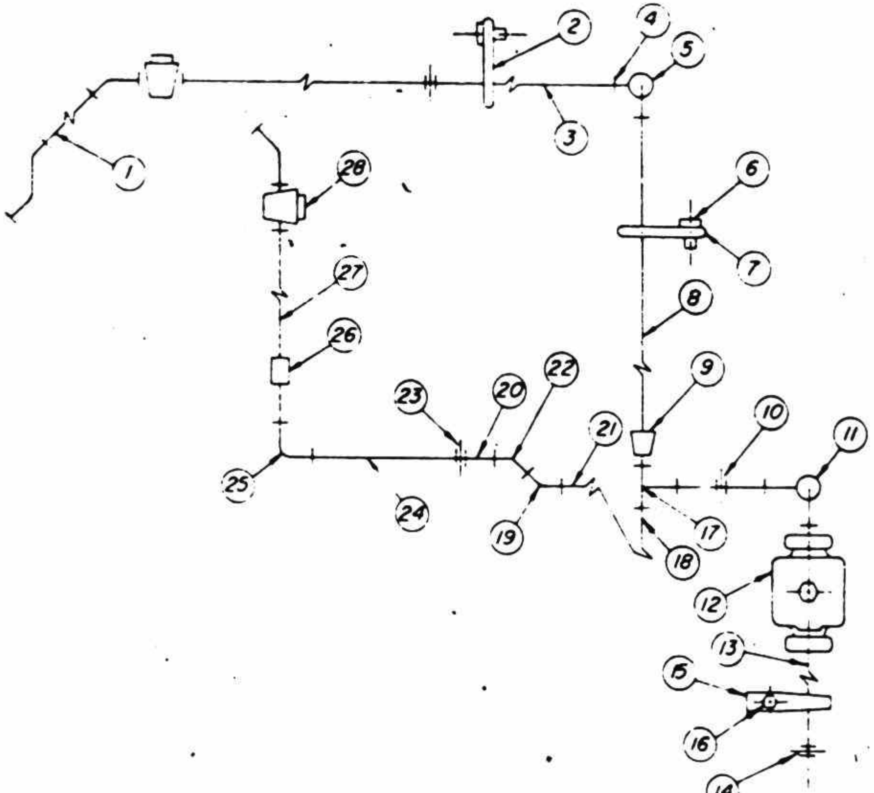
We must always keep in mind that the drawing must show an object which can in fact be constructed. Some difficulty was experienced in production of this eccentric triprong with ambihelical hexnuts. As noted on the bottom of the slide, no one has found a use for this object. If you look at the Widget carefully, you might notice that it changes in character. I'll leave the interpretation of this drawing to you - the main point we wish to convey to you is that, generally, someone else must be able to construct the object envisioned by the designer.

SLIDE NO. 5 (FIGURE NO. 5)

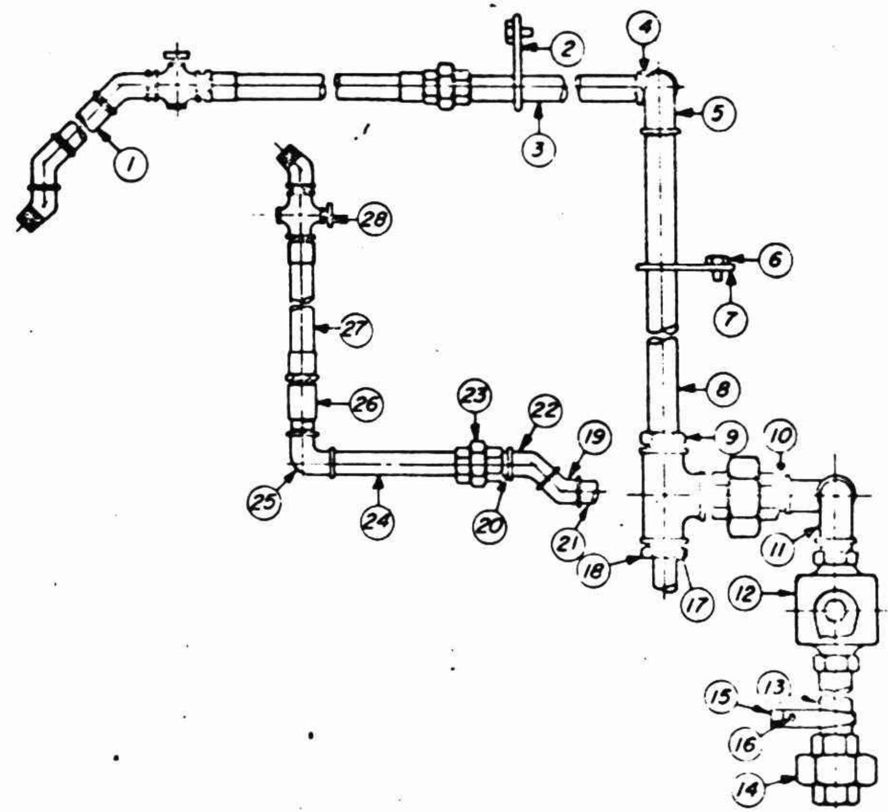
The designer of public works projects should be familiar with the field conditions of the site of the proposed works. A few minutes spent outside will not only be most informative but also give some appreciation of the problems of those who must construct the project. A single line on a drawing could represent a pipe line 6" to 18" in diameter and be readily drawn to miss all other objects. One must keep in mind the size of trench required to install such a pipe and allow not only for the pipe size but also the full width of the trench to avoid adjacent objects.

In viewing this slide, it can be seen that the field condition could be somewhat different than that envisaged by someone in the office who might be unfamiliar with the site. This trench has been designed for a trench condition which will be ideally suited for a particular pipe classification and type of bedding. However, although the drawing is

FIGURE NO. 6



Discount Drafting!



Redundancies, redundancies . . .

exaggerated, site and soil conditions may not allow for ideal design and possibly the pipe and pipe bedding should have been designed for an embankment condition. Embankment condition designs, as you will hear later this week, require stronger pipes and different bedding considerations.

In the second part of the slide we note that the bedrock denoted by the hatched lines, is nicely formed like a brick wall which will rarely be the case. The retaining wall has been designed to fit this natural configuration allowing the minimum of concrete for construction. No allowance for excavation or other backfill has been made. Here again, the site shown in reality exaggerated though it may be, gives quite a different picture. We must convey to others a more realistic idea of the problems which will arise during construction. Forming would be required on both sides of the wall adding to the cost; excavation would be required and allowance for granular backfill. We do not propose that your design would be predicated on either pattern shown here but drawn in a manner to indicate to others as nearly as possible the quantities of materials required and conditions to be met.

SLIDE NO. 6 - PIPING DIAGRAM (FIGURE NO. 6)

Although one should ensure that all the required information is on the drawing, as much attention should be given to ensure that redundancy is avoided. The two drawings shown here represent the same piping system. A bill of materials listed by number accompanies both drawings and consequently is not shown here since the time taken to prepare the list is the same for both. However, one can see that the line drawing could be done in 25% of the time taken for the detailed drawing and time means money.

Along similar lines, some designers like to have specifications written on the drawings involving much time wasted in lettering whereas probably all that is required is a specification reference. Speaking of

CHECK LIST - GENERAL PLANS

1. Scale 1" = 100'

2. Key Plan 1" = 1000'

3. North Point

4. Title Block
   a) Name of Subdivision or Project
   b) Completely signed - engineer's stamp if applicable
   c) Scale
   d) Date
   e) Table of Revisions

5. Reference Bench Mark

6. Existing services and details

7. Proposed services and details

8. Street names and street lines

9. Registered plan, lot and block numbers

10. Co-ordinate grid lines

FIGURE NO. 7

CHECK LIST - PLAN AND PROFILE

1. Scales - Horizontal 1" = 40' and Vertical 1" = 10'
2. North Point
3. Title Block
   a) Name of Subdivision or Project
   b) Completely signed - engineer's stamp if applicable
   c) Scale
   d) Date
   e) Table of Revisions
   f) Street name and limits
4. Reference Bench Mark
5. Existing Services and Details
6. Proposed Services and Details
7. Street Names and Street Lines
8. Registered Plan, Lot and Block Numbers
9. Co-ordinate Grid Lines
10. Building Basement Elevations
11. Station Chainages - Maximum Spacing 100'
12. Sewers
   a) Pipe Classification
   b) Bedding Type
   c) Gradient and Lengths
   d) Pipe Diameter
   e) House Connections
   f) Catch Basins and Connections
   g) Minimum Cover - 7' for storm
      - 9' for sanitary
   h) Manhole Details - Scale as Required
   i) Maintain 3" clearance between pipes at all crossings
   j) Drop structures used where drop is in excess of 3'0"
   k) Standard Drawing References
   l) Sewer Easements Widths
      - single sewer 20'
      - two sewers, single trench 25'
      - two sewers, separate trenches 30'
13. Roads
   a) Typical Finished Road Cross-Section
      i) Finished Surface Type
      ii) Base Courses
      iii) Crown
      iv) Curb or Curb & Gutter
      v) Pavement Width
      vi) Boulevard Treatment
   b) Curb Radii:-

Width of Pavement of Intersecting Streets	Curb Radius
28' x 28'	25'
28' or 32' & 42'	35'
28' or 32' & 50'	35'
42' & 42'	55'
42' & 50'	55'
50' & 50'	55'

   c) Road Grades - 28' pavement, maximum allowable 10.00%
      - over 28' pavement, maximum allowable 6.00%
      - minimum allowable 0.50%
      - Bulbs, cul-de-sac, turning circules, minimum allowable 1.00%
   d) Maximum Grade Change 2.00%
   e) Show Grade Change Symbols
   f) Distances and % Grades Between Grade Changes
   g) Catch Basin Spacing Maximum - 300'
      Road Grade over 5.00% - Maximum Spacing 200'
      Road Grade over 3.00% - use Side Inlet Catch Basins
      Low Points - Use Double Catch Basins
   h) Road Crowns:- 28' and 32' pavement widths - 4"
      42' and 50' pavement widths - 8"
   i) Station Chainages, Maximum Spacing - 100'
14. Other Special Details as Required in Design

FIGURE NO. 8

specifications, the designer should familiarize himself with the specs to ensure that the drawing and specs are complimentary and not contradictory.

SLIDES #'S 7 & 8 - CHECK LIST (FIGURES NOS. 7 & 8)

Since we are dealing here mainly with municipal engineering drawings, I have included a check list used in Scarborough of items which we consider necessary for a complete engineering drawing. This list is used not only for checking drawings of our own projects but also for checking drawings received from consulting engineers for subdivision construction. Many of you may have access to a similar list; others may not but we find the lists most useful.

The first check list is for checking the General Plan of a subdivision showing the details required. Under Items 6 & 7, the list could be expanded naming the actual services existing and proposed to be shown. For example, sewers & connections, watermains, valves & hydrants, curb & gutter, pavement, etc.

SLIDE NO. 8 (FIGURE NO. 8)

The second check list is for the plan and profile of each individual street and is much more detailed. To produce a complete drawing and design for a municipal project, we believe that a draftsman should refer to this ready reference list and in fact all draftsmen and plan examiners should have a copy.

USES FOR DRAWINGS

One of the more frequent problems we have encountered is the lack of understanding by the draftsman of the purpose for which a drawing is intended. This misunderstanding is frequently caused by the person requesting the drawing in that insufficient information has been provided and a communication gap exists.

For instance, if you are asked to produce a line drawing for presentation to a committee of Council or a ratepayers' group, you must

envisage the weight of lines required governed by the distance between the speaker and the audience. The size of lettering must be exaggerated so that it can be read easily from 20 or 30 feet away.

WATERCOURSE MAP OF SCARBOROUGH

This drawing was prepared for demonstration purposes at this lecture. I trust it is readable from your vantage point. Usually such a drawing will not be too detailed but emphasize only the highlights to be considered. In this case, I asked for a map showing the three major watercourses in Scarborough indicating completed sections with a heavy solid line, incompleted sections with a heavy dashed line and the year for completing the work up to 1976.

On the other hand, if you are asked for a line drawing for a committee report, immediately the size is determined as $8\frac{1}{2}$" x 11" (letter size) since the practice is to reproduce copies of the drawing for each copy of the report distributed to interested parties. The word "line" indicates elimination of unnecessary details, perhaps related to the drawing but not of sufficient importance on which to waste drafting time.

The lines and dates shown here define the major topic under consideration. At this stage we are interested in main watercourses only and not in minor sewers flowing into the main stream. Additions of a minor nature would serve only to confuse the main issue.

Sometimes designers and draftsmen are required to make freehand sketches envisaging the first concept of the project. Although these are widely used in the architectural field for buildings, etc., there is a place for such drawings in municipal engineering. Unfortunately not many people appear to have the artist's gift of producing such drawings but, with practice, this can be accomplished and subsequent drawings will be easier to produce.

Plan and profile drawings for tender purposes require much more thought, accuracy and completeness than the foregoing and, in general, comprise 75% or more of the drawings produced in municipal offices. As previously mentioned, it is the practice in our office to indicate in heavy lines the proposed works and in lighter lines, existing features. In this way it is easier for those using the drawing to interpret the work to be done.

The check lists noted earlier cover fairly well the requirements for the municipal street type of plan & profile and need not be repeated here.

Finally, many municipalities have a set of "Standard Drawings" which are drawings of units such as manholes, catch basins, sidewalks, etc., which will be constructed in the same manner many times. To redraw such plans on every occasion would be time-consuming and costly. There is currently a committee set up to develop a set of Standard Drawings or preferably "Model Municipal Designs" for municipalities in Ontario and, hopefully, these will be available by 1972.

FIELD INFORMATION

The main line of communication between the draftsmen in the office and the survey crews in the field is the field survey notes. In general, engineering offices employ surveyors to carry out field survey work and draftsmen for office work, except perhaps in smaller offices where a few people fulfil both functions. The people involved in the separate functions tend to become specialists in one field and lose sight of the total operation. The instruction from the office personnel to the surveyors should be clear and concise with an explanation of the purpose for the survey, the detail required and possibly a date for completion of the survey. In order to issue such instructions, it is imperative that draftsmen should be familiar with field practices, have some knowledge of how the field notes are produced and know the mathematics involved in reduction of the notes.

Since most municipal engineering drawings are of the plan and profile type of municipal streets, we will consider those first.

Initially, field crews must search for evidence in the form of survey bars marking street line limits. On many occasions this can be a time-consuming and frustrating experience, especially in older areas where most of such evidence has been destroyed. The draftsman or designer can plot such evidence in a few minutes but he should recognize that quite often finding this evidence may take the field crew a day of searching. Delays in obtaining the surveys required are often caused in this way.

SLIDE NO. I - ESTABLISHING CENTRE LINE (FIGURE NO. I)

Assuming that sufficient evidence has been found, the surveyor establishes the legal centre line of the road by offsetting half the width of the road allowance from the bars.

The draftsman begins the plan layout in a similar manner. The first item to be drawn is usually three parallel lines representing the two street lines and the centre line. Although the centre line so estab-

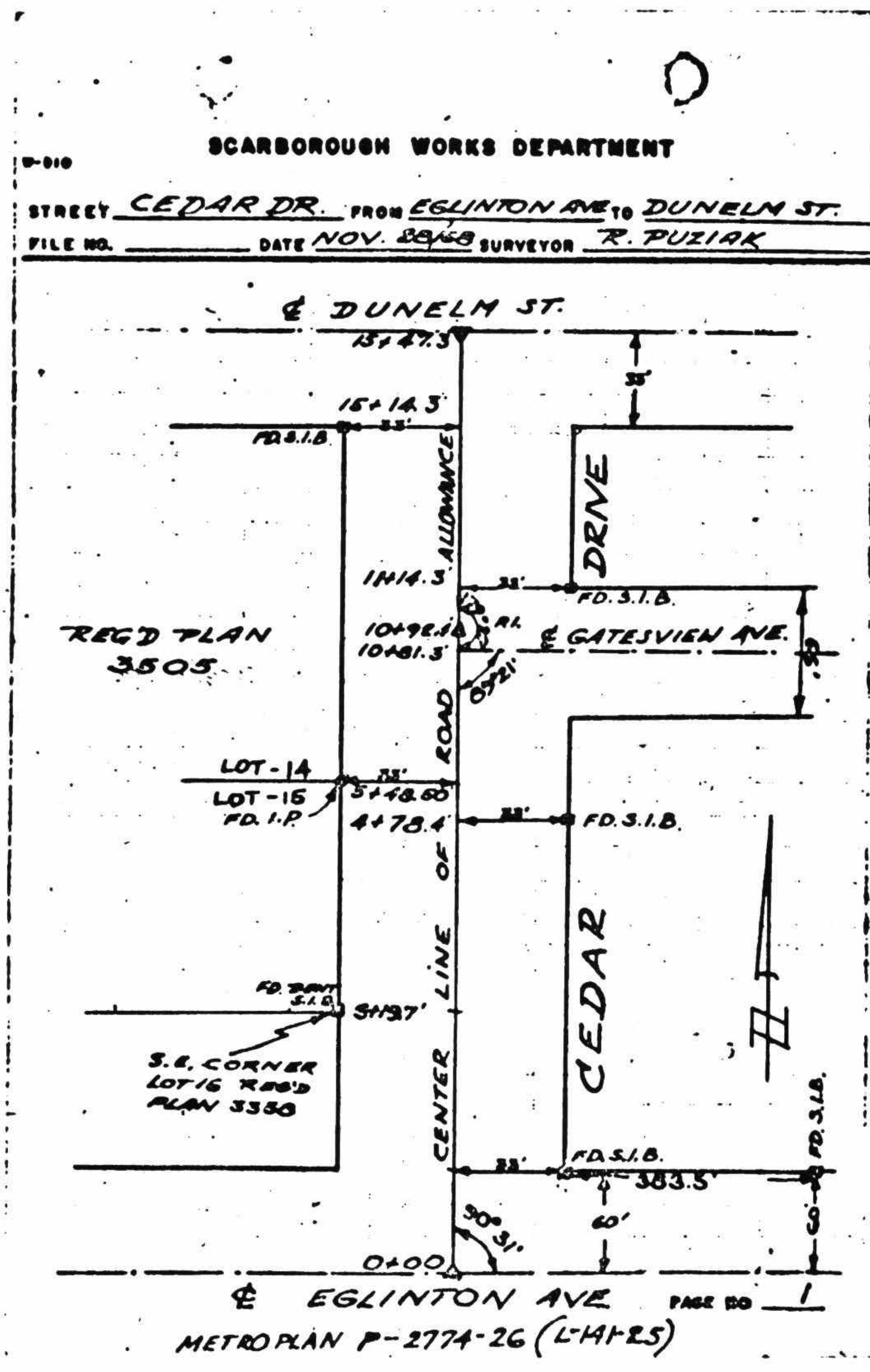

SCARBOROUGH WORKS DEPARTMENT
STREET CEDAR DR. FROM EGLINTON AVE TO DUNELM ST.
FILE NO. DATE NOV. 28/68 SURVEYOR R. PUZIAK
℄ DUNELM ST.
15+47.3
15+14.3
FD.S.I.B.
DRIVE
ROAD ALLOWANCE
11+14.3
FD.S.I.B.
REG'D PLAN 3505
10+92.1
10+81.3
℄ GATESVIEW AVE.
LOT-14
LOT-15
FD. I.P.
5+48.60
4+78.4
FD.S.I.B.
CENTER LINE OF
CEDAR
S.E. CORNER LOT 16 REG'D PLAN 3350
FD.S.I.B.
383.5
60'
0+00
℄ EGLINTON AVE
PAGE NO 1
METRO PLAN P-2774-26 (L-141-25)

FIGURE NO. I

lished is normally used, on streets carrying large volumes of traffic, offsets are sometimes made in the boulevard for control survey lines and the draftsman should be alert for such eventualities. It is all too easy to overlook the exception to the rule when working on a drafting board all day.

In Scarborough, we generally use the intersection of the centre lines of two streets as the 0+00 chainage point and progress either northerly or easterly from that point. This convention is used so that your drawing may be properly oriented with the north point in the first quadrant of your Cartesian co-ordinate system. In plotting the 0+00 point on your drawing, ensure that this point can be projected vertically to a main division line on your profile view. For straight streets, this makes plotting of the profile much easier. On curved streets, of course, the profile will not project vertically downward but 0+00 should still be plotted on a major division line on the profile.

The slide we are viewing was taken directly from field notes made in Scarborough in 1968. Chainages are given at intersecting streets - i.e. 10 + 81.3 at Gatesview Avenue and 15 + 47.3 at the centre line of Dunelm Street. The angles of intersection at Eglinton Avenue and Gatesview Avenue are also noted. It is the responsibility of the drafts-man to check the registered plan, in this case Nos. 3505 and 3338, to ensure that the field notes and registered plan agree in these dimensions. Although this page of field notes appears quite simple, there is a wealth of information given here for drafting the plan view. Chainages are shown at Lot 16, Registered Plan 3338, the division line between Lots 14 and 15, Registered Plan 3505, the south limit of Dunelm Street and the north limit of Gatesview Avenue. There is also a plan reference given for details of Eglinton Avenue, a 120-foot wide road. It should be remembered that field notes are not to scale but measurements should be plotted as

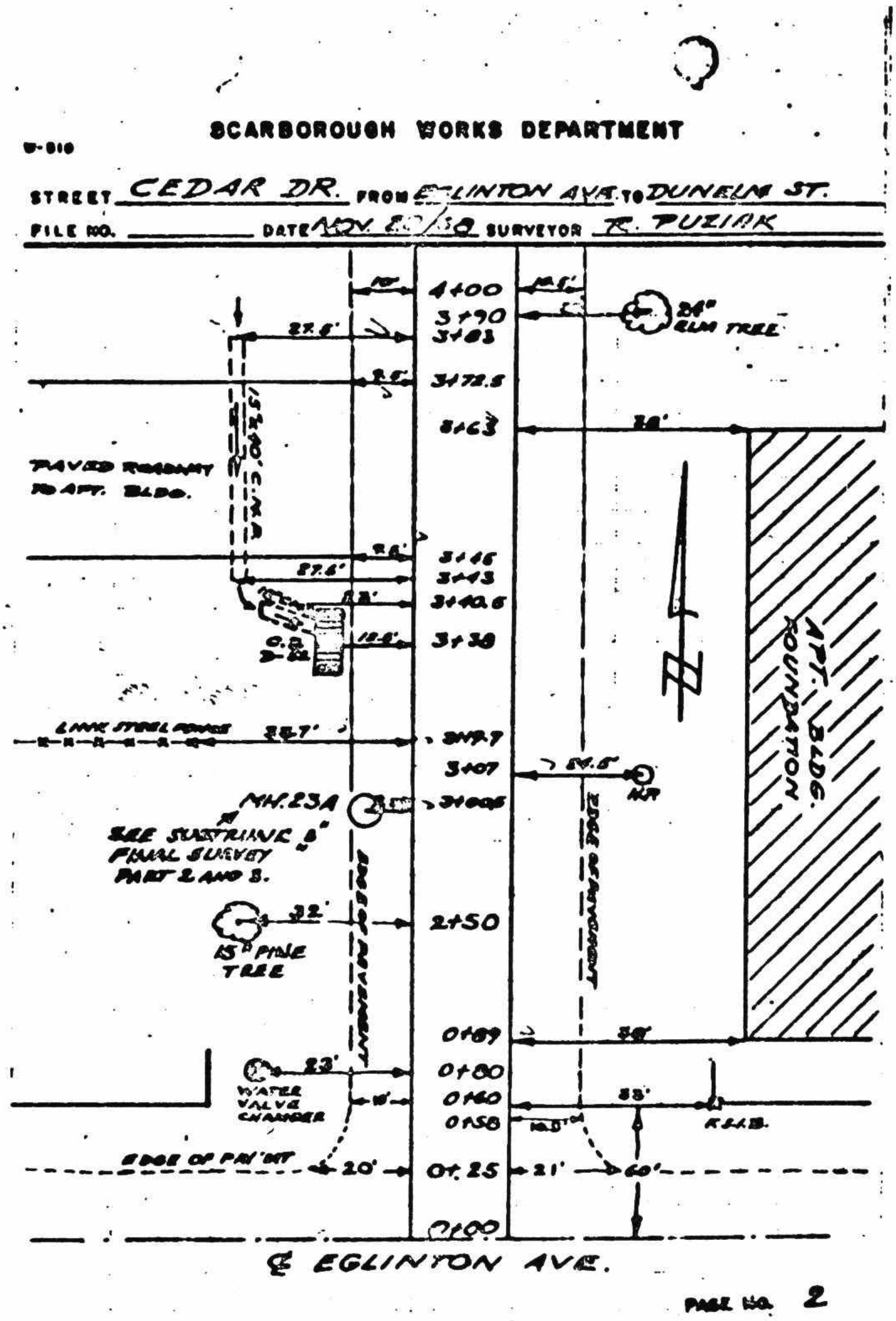

SCARBOROUGH WORKS DEPARTMENT
STREET CEDAR DR. FROM EGLINTON AVE. TO DUNELM ST.
FILE NO.
DATE
SURVEYOR R. PUZIAK
4+00
3+90
3+83
3+72.5
3+63
3+46
3+43
3+40.6
3+38
3+07
2+50
0+89
0+80
0+60
0+58
0+25
0+00
24" ELM TREE
PAVED ROADWAY TO APT. BLDG.
APT. BLDG. FOUNDATION
MH.23A
15" PINE TREE
WATER VALVE CHAMBER
EDGE OF PAV'MT
20'
21'
60'
32'
23'
27.6'
€ EGLINTON AVE.
PAGE NO. 2

FIGURE NO. 2

shown. Any errors can readily be detected at this early stage.

SLIDE NO. 2 (FIGURE NO. 2)

Having established the street centre line and the 0+00 control station, the surveyor proceeds to tie in plan detail. There are two commonly used methods for plotting plan detail in survey notes. Some surveyors use a single centre line as on Slide No. 1 showing centre line chainage and the distance from the centre line to the reference object. Using a right-angle prism, accurate right angles off the centre line can be quickly measured. The right-angled measurement from the control centre line again makes the draftsman's job much easier.

The second method of drawing field notes for plan detail is carried out by the use of a split centre line as shown in this slide. In other words, two parallel lines are drawn about 3/4" apart, each representing the centre line. The space between will contain the station such as 3+38 and dimensions to objects off the centre line are plotted at right angles to either side. We use this method since it more readily allows for measurements for two objects either side of centre at the same station.

Abbreviations and symbols are essential for both field notes and plans and the same abbreviations and symbols should be used for both purposes. Adoption of the Metropolitan Toronto Public Utilities Co-ordinating Committee Legend would assist in interpretation of field notes if used both in the field and in the office. Many symbols or abbreviations are well known to all of us such as M.H. for manhole, C.B. for catch basin, S.I.B. for standard iron bar but how about C.I.P.? Is this corrugated iron pipe or cast iron pipe? And S.S. - is this storm sewer, street sign, sanitary sewer, stop sign, etc? Surveyors have a tendency to use more abbreviations than do draftsmen due to the limited size of paper they use. Make sure that a communication gap does not exist in this area.

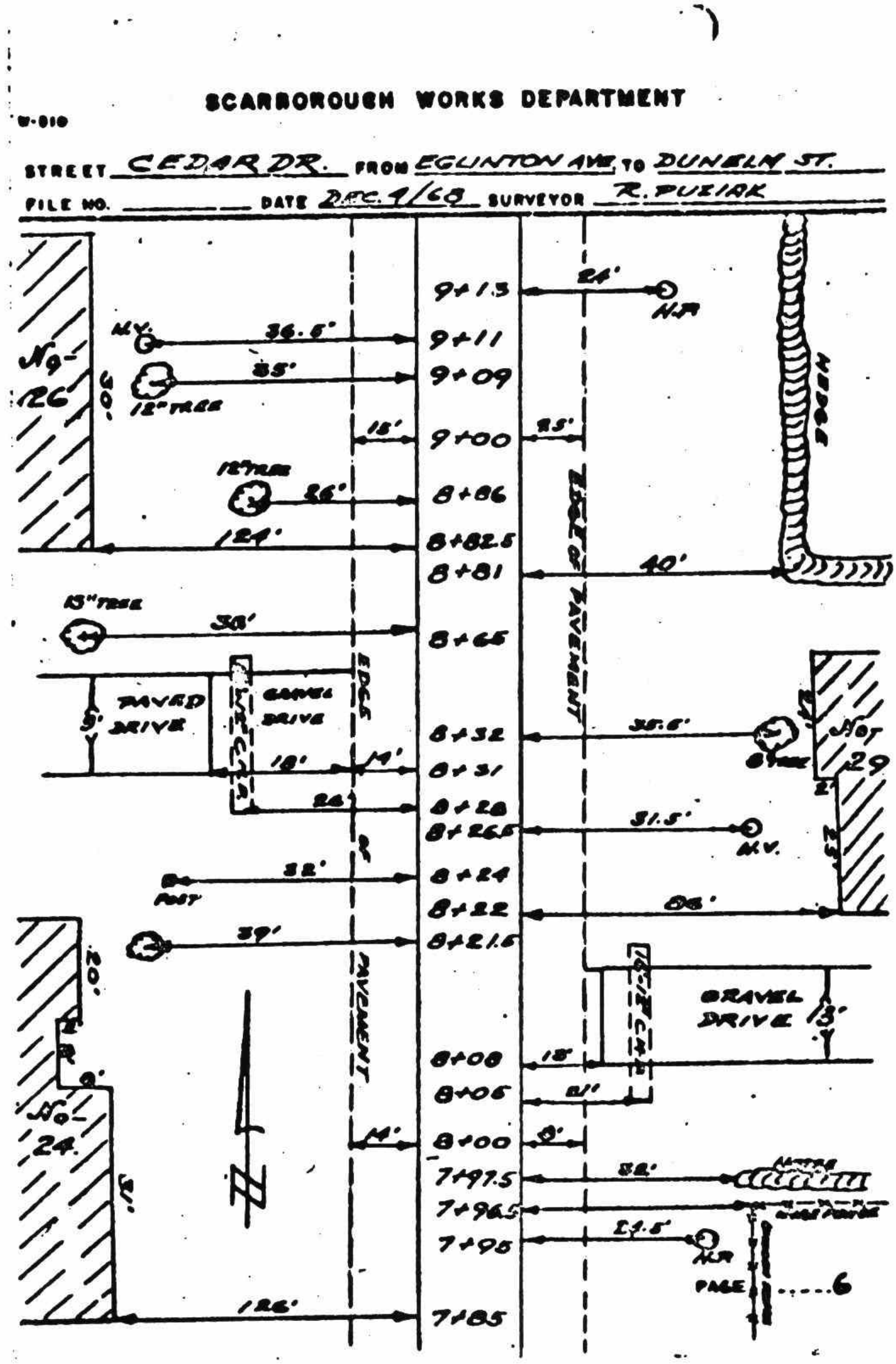
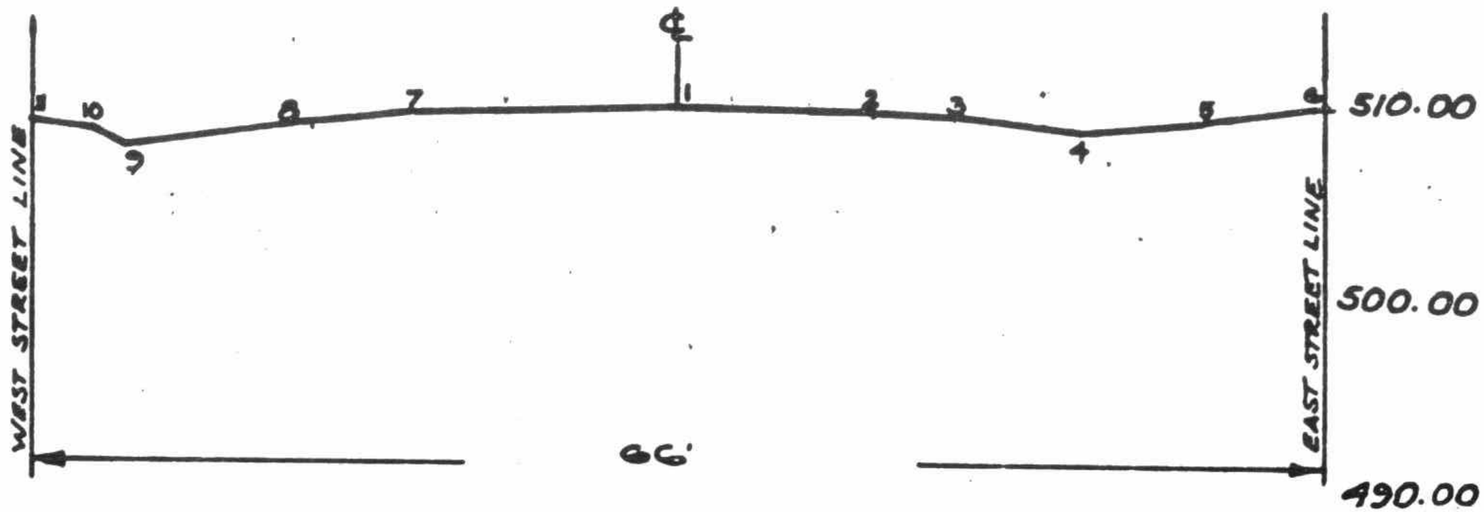

SCARBOROUGH WORKS DEPARTMENT
STREET CEDAR DR. FROM EGLINTON AVE TO DUNELM ST.
FILE NO. DATE DEC. 4/68 SURVEYOR R. PUZIAK
9+13
9+11
9+09
9+00
8+86
8+82.5
8+81
8+65
8+32
8+31
8+28
8+26.5
8+24
8+22
8+21.5
8+08
8+06
8+00
7+97.5
7+96.5
7+95
7+85
HEDGE
EDGE OF PAVEMENT
PAVED DRIVE
GRAVEL DRIVE
GRAVEL DRIVE
12" TREE
POST
PAGE 6

FIGURE NO. 3

FIGURE NO. 4



CEDAR DRIVE

FROM EGLINTON AVE. TO DUNELM ST.

TYPICAL CROS-SECTION OF ROAD AT STA. 6+00

SCALE 1" = 10'-0"

SLIDE NO. 3 (FIGURE NO. 3)

A survey for the plan detail should show ties to all visible objects at least from street line to street line and preferably to buildings. Dimensions to buildings are mandatory when carrying out a survey for sewer works since it is essential to have the distance from the building to the main sewer for calculation not only of the sewer connection gradient but also of the depth required for the main sewer. This particular survey was a preliminary survey for a proposed sewer. For most surveys, it is necessary to tie in the location of all driveways so sewer connections can be designed off the driveways. In fact, our present practice is to do a complete survey for all purposes at one time to avoid repetition of work. This slide shows building locations and dimensions, hydro poles, hedges, driveways, culverts, trees and trunk diameters, edge of pavement, etc.

SLIDE NO. 4 - CROSS-SECTION (FIGURE NO. 4)

The field notes should contain a typical cross-section of the road to be surveyed from street line to street line. Approximate elevations are shown at the sides for plotting end areas for the calculation of quantities for road improvements. The numbers 1 to 11 indicate positions of the elevations to be taken at each change in grade of the cross-section. This gives the draftsman a good idea of the existing cross-section. In this case, the road appears to be very flat with poorly defined ditches, shoulders, etc. Cross-sections on other streets may be radically different than the one shown here and have curbs, sidewalks, deep ditches, high crowns, narrow travelled sections, etc.

SLIDE NO. 5 (FIGURE NO. 5)

<u>LEVELS</u>

Many of you may have some survey experience. However, for the benefit of those who have not, we will examine a page of level notes.

U-800 **SCARBOROUGH WORKS DEPARTMENT** PAGE NO. 14

DATE DEC. 12/68 STREET CEDAR DRIVE FROM EGLINTON AVE. TO DUNELM ST. SURVEYOR T. PUZIAK

STATION	B.S.	H.I.	I.S.	F.S.	ELEV.	REMARKS
	3.00	509.81			506.81	TOP OF HYD ON E. SIDE OF CEDAR DR. 1ST N. OF EGLINTON AVE.
5+00			2.65		507.16	33'W. TOP DITCH
			2.75		507.06	7'E. EDGE OF PAVEMENT
			2.85		506.96	15'E TOP DITCH
			3.55		506.26	22'E ℄ DITCH
			2.45		507.36	30'E. TOP DITCH
			2.00		507.81	33'E ⅊
5+50			1.30		508.51	℄ RD.
T.P.	9.42	517.04		2.19	507.62	
6+00			7.50		509.54	℄ RD. (1)
			7.70		509.34	10'E. EDGE OF PAVEMENT (2)
			7.90		509.14	14'E. TOP DITCH (3)
			8.55		508.49	20.5'E ℄ DITCH (4)
			7.95		509.09	27'E. TOP DITCH (5)
			7.45		509.59	33'E ⅊ (6)
			7.90		509.14	13.5'W. EDGE (7)
			8.30		508.74	19.5'W. TOP DITCH (8)
			9.75		507.29	28'W ℄ DITCH (9)
			8.85		508.19	30'W. TOP DITCH (10)
			8.40		508.64	33'W. ⅊ (11)
6+50			6.40		510.64	℄ RD.
				3.01	506.80	B.M. -0.01

FIGURE NO. 5

All draftsmen and designers should be able to reduce level notes and check them before plotting. In some offices these calculations are done only by the draftsmen and in others by the surveyors.

The following headings are used:

Station - Denoting the distance measured along the centre line or control line from 0+00.

Backsight - is the rod reading taken from the Bench Mark to establish the height of instrument.

H.I. - The height of the instrument is obtained by adding the rod reading of the backsight to the elevation of the Bench Mark.

I.S. - Intermediate Sight is a rod reading taken at any given point, the description of which is given in the right-hand column and for which the elevation is calculated by subtracting this rod reading from the height of instrument. The result is then recorded under the elevation column.

F.S. - Or foresight is a rod reading taken on a turning point prior to moving the instrument to a new location. Subtract this reading from the height of instrument to obtain the elevation of the turning point.

T.P. - The turning point should be taken on some solid object not subject to disturbance and upon which an elevation for temporary use can be established. The instrument can then be moved to a new location from which another reading is taken to establish a new height of instrument. It is mandatory to return to the original Bench Mark as a check on the accuracy of the survey. In this case the error is 0.01 feet which is quite acceptable.

FIGURE NO. 6

PAGE NO. 21

U-800

SCARBOROUGH WORKS DEPARTMENT

DATE DEC. 16/68 STREET CEDAR DRIVE FROM EGLINTON AVE TO DUNELM ST. SURVEYOR R. PUZIAK

STATION	B.S.	H.I.	I.S.	F.S.	ELEV.	REMARKS
B.M. #1	4.95	511.76			506.81	TOP OF HYD. E/S. CEDAR DR. 1ST N. OF EGLINTON AVE.
HOUSE #12			6.36		505.40	S. SILL-4.56 BASEMENT ELEV. 500.84
-"-			6.25		505.51	DR. @ ℄
-"-			6.45		505.31	-"- 10'W. OF ℄.
APT. BLDG.			15.13		496.63	BASEMENT FLOOR.
House #27	INACCESSIBLE		DUE	TO (2)	VICIOUS	DOGS.
HOUSE #14			4.05		507.71	S/W. SILL 4.65 BASEMENT ELEV. 503.06
-"-			5.80		505.96	DR. @ ℄
-"-			5.90		505.86	-"- 10'W. OF ℄.
-"- #19			0.20		511.56	S/W. SILL-4.65 506.91
-"-			3.35		508.41	DR. @ ℄
-"-			2.90		508.86	-"- 10'E. OF ℄.
T.P.	10.66	518.25		4.17	507.59	
HOUSE #21			4.06		514.19	S/W. SILL-4.58 509.61
-"-			9.25		509.00	DR. @ ℄
-"-			9.05		509.20	-"- 10'E. OF ℄.
HOUSE #18			9.58		508.67	S/E SILL-4.73 503.94
-"-			10.00		508.25	DR. @ ℄
-"-			10.50		507.75	-"- 10'W. OF ℄.
T.P.	7.07	517.80		7.52	510.73	

<u>Elevation</u> - For recording the elevation of all points.

<u>Remarks</u> - This column is used to describe the point where the elevation is taken, the numbers 1 to 11 referring to the typical cross-section given in the previous slide.

SLIDE NO. 6 (FIGURE NO. 6)

For sewer surveys, in addition to having the longitudinal profile and cross-section of the street, it is necessary to establish the basement elevation of the houses and other buildings. The practice in Scarborough is to take the basement window sill elevation and go into the building and measure from the sill to the basement floor to establish the basement floor elevation. From this elevation for sewer design purposes, it is necessary for gravity flow to allow an additional 30" below the floor level and to allow for a 2% minimum grade for the sewer connection from the building to the proposed sewer. All is not easy on the outside as you can see by the remark made opposite House #27, "Inaccessible due to 2 vicious dogs." As I mentioned earlier, these notes were taken directly from our files and not changed in any way for this presentation except to ink them over so that clear reproductions could be made.

Generally, road cross-sections are taken at even intervals of either 50 or 100 feet depending on the uniformity of the existing road gradient and character of its cross-section. However, the surveyor will on occasion take additional sections to show unusual features. All of the sections should be plotted by the draftsmen for determining quantities. The survey notes will show such objects as watermain valves, gas valves and Bell manholes but not the route of the lines between them. It is the responsibility of the draftsman and designer to search for this information from the office files and plot the utility lines.

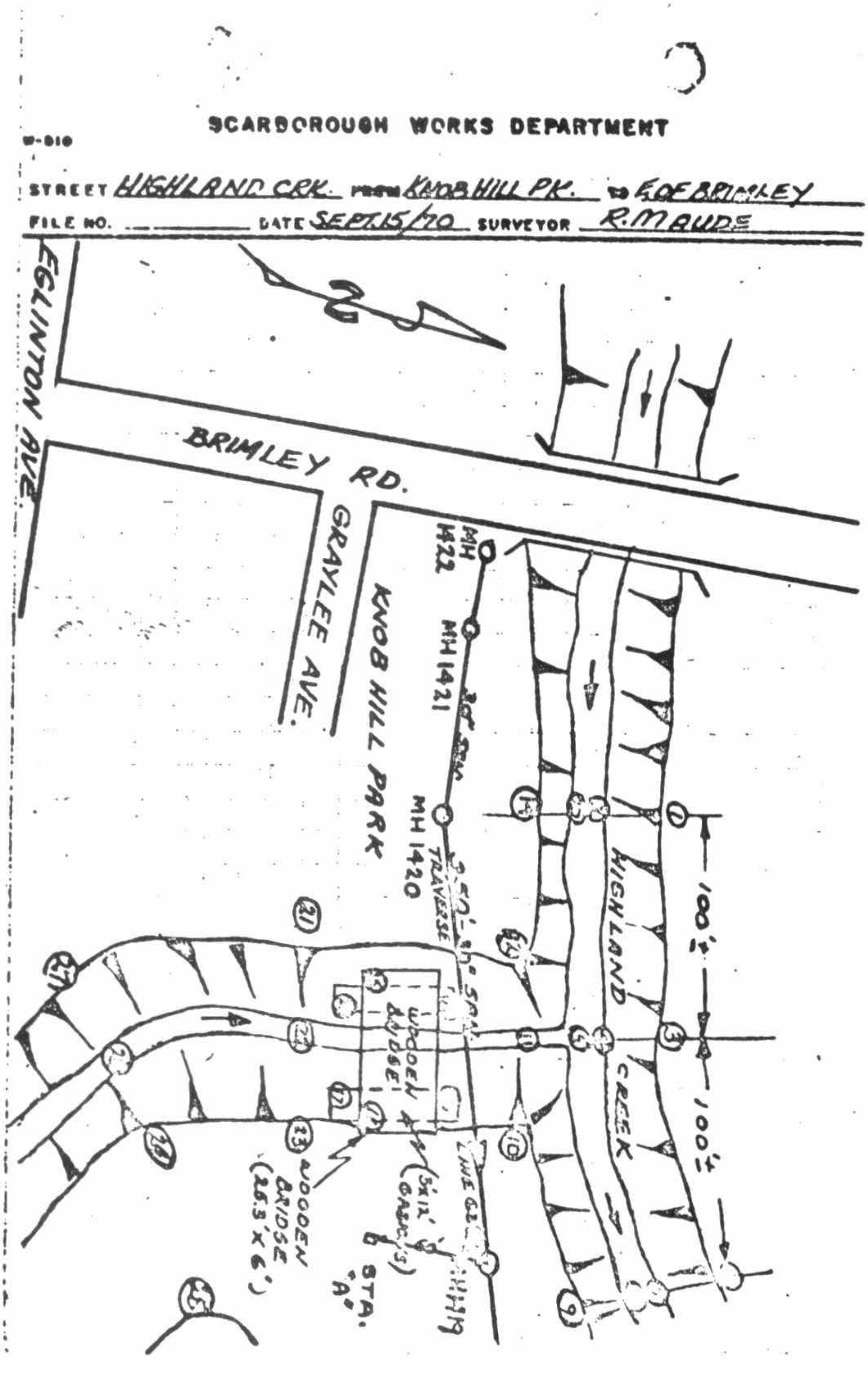

SCARBOROUGH WORKS DEPARTMENT
W-810
STREET HIGHLAND CRK. FROM KNOB HILL PK. TO E. OF BRIMLEY
FILE NO.
DATE SEPT. 15/70
SURVEYOR R. MAUDE
EGLINTON AVE.
BRIMLEY RD.
GRAYLEE AVE.
KNOB HILL PARK
MH 1422
MH 1421
MH 1420
TRAVERSE
HIGHLAND CREEK
100'±
100'±
WOODEN BRIDGE
WOODEN BRIDGE (25.3' x 6')
STA. "A"
21
27

FIGURE NO. 7

STADIA FIELD NOTES

BOROUGH OF SCARBOROUGH

WORKS DEPARTMENT

STREET HIGHLAND CRK FROM [illegible] TO E OF BRIMLEY

FILE No. ____ PLAN No. ____ DATE [illegible] SURVEYOR R. MAUDE

STATION	H. C. R.	ROD INT	V. C. R.	HOR DIST.	DIFF. ELEV	ELEV.	REMARKS
Ⓧ MH 1419						497.74	W. LUG MH 1419
B.S. MH 1420				E.I.	=502.6?		B.S. MH 1420 LINE & GRADE
H.I. 4.73	18° 20'	164'	-2° 25'	163.8	-6.91	491.05	(1) TOP BK. 100' W OF JUNC.
"	17° 50'	164'	-3° 13'	163.6	-9.12	488.75	(2) BOTT BK. " " "
"	42° 20'	71'	-5° 27'	70.3	-6.71	491.23	(3) TOP BK. OPP. SMALL CRK.
"	41° 00'	70'	-7° 02'	69.0'	-8.61	489.33	(4) BOTT " " "
"	38° 29'	69'	-7° 35'	67.9'	-9.04	486.90	(5) JUNCTION ℄ CRKS.
"	131° 40'	79'	-3° 12'	78.8'	-4.40	493.54	(6) TOP BK. 100' E OF JUNC.
"	137° 40'	72'	-7° 25'	70.7'	-9.22	488.72	(7) BOTT " " " "
"	145° 20'	[illegible]	-8° 28'	63.5'	-9.46	488.48	(8) BOTT " " " "
"	150° 00'	62'	-5° 0?'	61.5'	-5.45	492.49	(9) TOP " " " "
"	30° 40'	60'	-5° 32'	59.4'	[illegible]	492.18	(10) TOP S.E. BK @ JUNCT.
"	31° 00'	69'	-8° 14'	67.1'	-9.78	488.16	(11) ℄ CRK. (SMALL)
"	28° 20'	72'	-5° 31'	71.3'	-6.89	491.05	(12) TOP S.W. BK @ JUNCT.
"	14° 48'	160'	-3° 12'	159.6'	-8.92	489.02	(13) BOT. BK. 100' W. OF JUNC.
"	13° 50'	159'	-2° 22'	158.7'	-6.56	491.38	(14) TOP BK. " " "
"	341° 00'	108'	-1° 23'	107.9'	-2.60	495.34	(15) S.W. CORN. BRIDGE
"	329° 20'	95'	-1° 26'	94.9'	-2.36	495.58	(16) S.E. " "
"	332° 20'	100'	-2° 13'	99.9'	-3.86	494.08	(17) CORN. GABION
"	335° 48'	90'	-2° 24'	87.9'	-3.77	494.17	(18) " "
"	342° 48'	98'	-2° 25'	97.8'	-4.13	493.81	(19) " "
"	338° 41'	107'	-2° 07'	106.9'	-3.95	493.99	(20) " "
"	338° 40'	114'	[illegible]	113.9'	-3.12	494.82	(21) TOP CRK. BK @ BRIDGE
5.73	339° 20'	108'	-3° 49'	107.5'	-7.17-1	489.77	(22) ℄ CRK. " "
4.73	328° 20'	101'	-1° 27'	100.9'	-2.61	495.33	(23) TOP CRK. BK.
"	317° 40'	150'	-1° 22'	149.9'	-3.7?	494.16	(24) " " " -BEND
"	300° 20'	145'	+ 2° 00'	144.8	+5.06	503.00	(25) TOP HIGH BANK

16-610

FIGURE NO. 8

SLIDE NO. 7 - TOPOGRAPHICAL SURVEYS (FIGURE NO. 7)

Topographical Surveys can be used in many ways. However, one major application in municipal work is surveys by stadia for watercourse improvements.

The stadia method is used due to the extent of the watercourse which may be from 100 to 200 feet from top of one bank to top of the opposite bank and also due to the rapid changes in section that may occur in ravines. The elevation changes from the invert of a stream to the top of the bank may be 100 feet in some cases and it is not economically feasible to take cross-sections in the same manner as for road work with ordinary levels and chains.

This slide shows a plan view of the watercourses on which the numbers 1 to 27 indicate the locations and elevations of points that were taken by stadia.

SLIDE NO. 8 (FIGURE NO. 8)

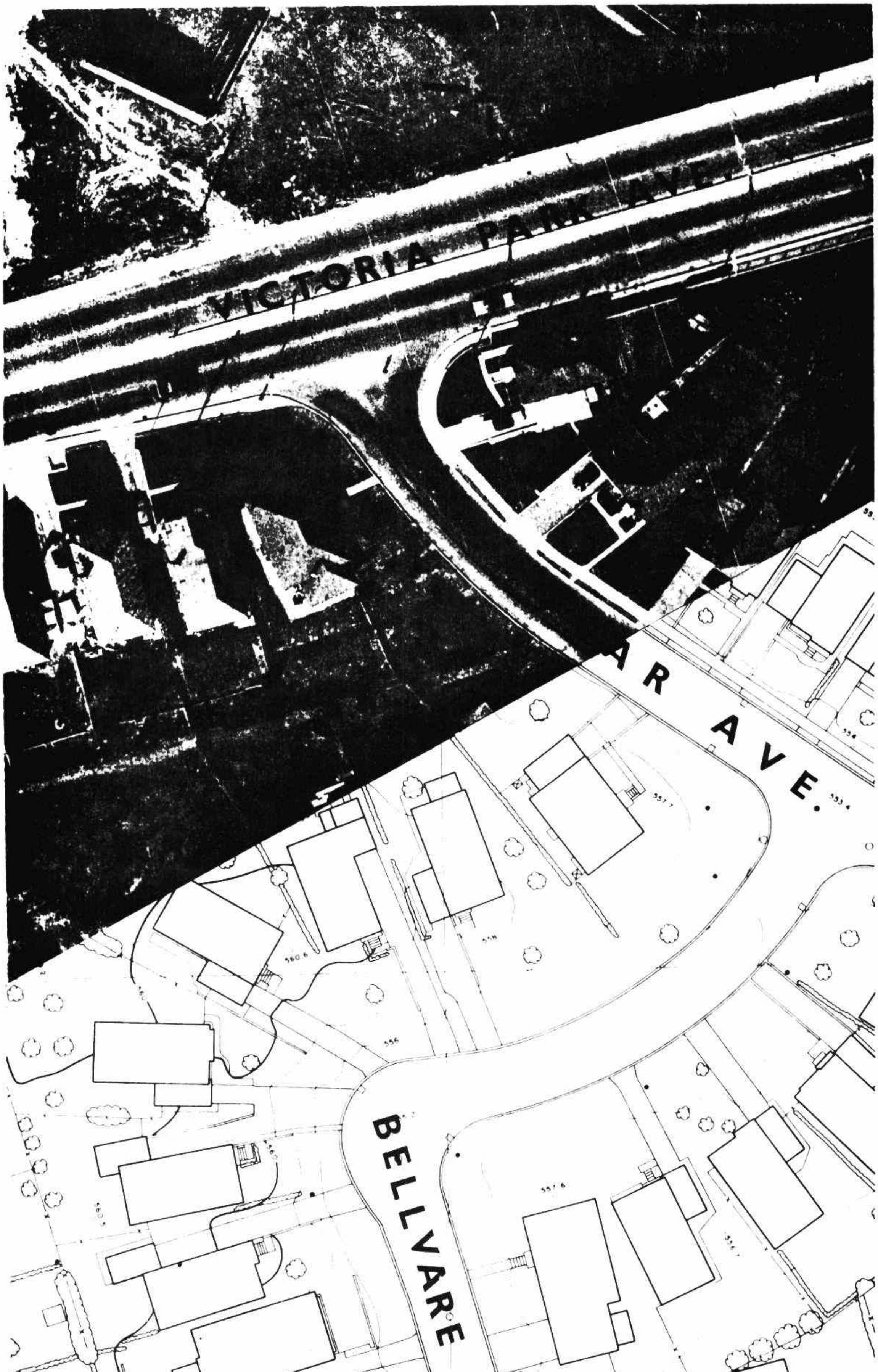
On slide #8 under remarks, we have the points listed again along with the description of the point taken. The base line is noted as a line from Manhole 1419, elevation 497.94, to manhole 1420.

The height of instrument is established in a different manner for stadia surveys than was previously noted for other types of surveys. The actual vertical distance from the top of the manhole 1419 to the central axis of the instrument is measured and added to the manhole elevation - in this case 4.73 + 497.94 = 502.67.

H.C.R. or horizontal angles are measured clockwise from the base line to the object being located.

Rod interval is the difference read between the top and bottom cross-hairs on the instrument and multiplied by 100 to obtain the distance to the object.

The vertical angle is read having sited 4.73 on the rod with the centre horizontal cross-hair on the instrument.


VICTORIA PARK AVE.
AR AVE.
BELLVARE AVE.

Stadia note reductions are calculated from these readings using stadia tables giving corrected horizontal and vertical measurements. Draftsmen should be able to calculate and check stadia notes. Plotting is then done by angle and distance from the control lines which may be established by conventional traverses or stadia traverses. The conventional method is more accurate for the base line control and reduces the inherent error of stadia surveys.

SLIDE NO. 9 - AERIAL PHOTOGRAPHIC MAPPING (FIGURE NO. 9)

To obtain more accurate topographical mapping of large or small areas, especially where the elevation of the land changes rapidly, aerial photography is the most effective and most economical means. From aerial photos, plotting of the maps is done initially by machine and subsequently "Fair Drawn" by hand.

There are many advantages to this method of mapping.

1. Economy of production - the cost is about $30.00 per acre.
2. Much more detail can be obtained.
3. No disturbance to owners of the lands.
4. Inaccessible places can be mapped.
5. Greater speed in production.
6. Contours of terrain such as Scarborough Bluffs can be obtained which is virtually impossible by other means.
7. All maps are related to one co-ordinate grid.
8. Both horizontal and vertical dimensions can be obtained on one map.
9. North is always to the top of the page.

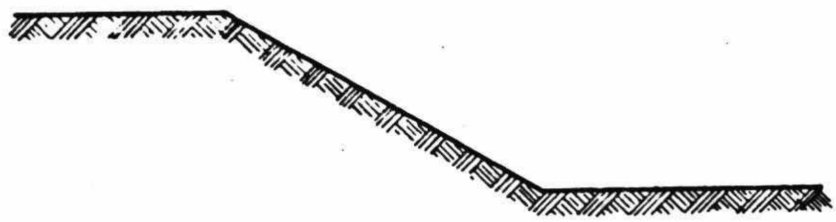
There are a few major disadvantages and these are that no underground information is obtained and for underground intallations, profiles must be obtained by conventional means.

Imaginary lines such as lot lines, street lines, etc., must be

superimposed on the aerial maps by conventional means. However, for delineating drainage areas or watersheds, we have found the aerial mapping most valuable. The mapping can be obtained at whatever scale you wish, some of the more common being 1" = 40', 1" = 200' and 1" = 400'.

SLIDE NO. 10 - DRAFTSMAN'S DELUSIONS (FIGURE NO. 10)

As noted in our earlier discussions, along with interpretation of field notes, a visit to the site by the designer is usually a good idea. As can be seen from this slide, the actual and the draftsman's concept might be far apart. We do not propose that you draw such exaggerations as are shown here but make an attempt to interpret the field notes in a realistic manner.

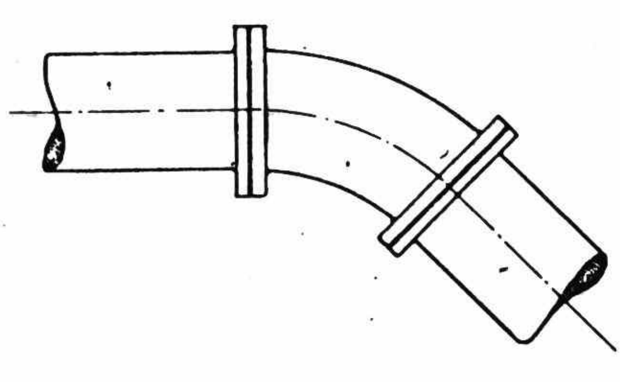


GROUND SURFACE

(DRAWING OFFICE)

GROUND SURFACE

FIELD



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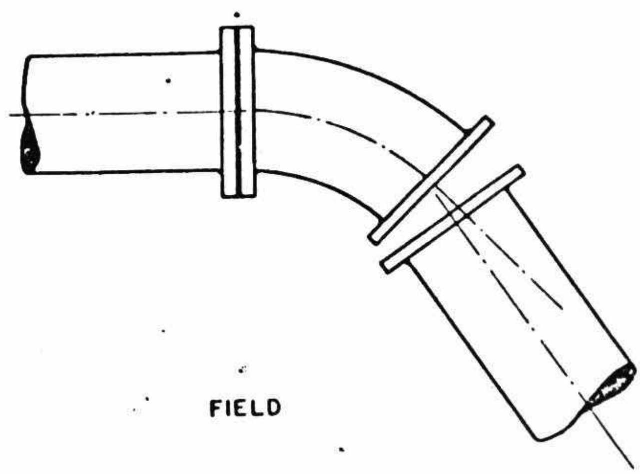


FIELD

DRAFTMAN'S DELUSIONS

FIGURE NO. 10

DESIGN ASPECTS

OF

A WATER DISTRIBUTION SYSTEM

by

Mr. A. Fields, P. Eng.

and

Mr. F. Booth, P. Eng.

CITY ENGINEERS ASSOCIATION DESIGN COURSE

WATER MAIN DESIGN

By: Mr. A. Fields

All water systems are composed of three main parts: transmission facilities to transport water from the source to the distribution mains, distribution mains to transport water from the transmission mains to the service lines, and service lines to transport water from the distribution grid to the customer.

In this session we will examine the factors involved and procedures to be followed in the design of transmission and distribution mains.

SELECTION OF ROUTE

In most towns and cities, utility distribution facilities are in public right-of-way. The trend has been to place sewer, gas, telephone, electric and drainage lines underground. The resulting crowded situation, especially in urban centres, requires that the design engineer determine the location of all existing services and proposed installations by other utilities. This is usually carried out by submitting a street line plan to the various utilities requesting that they indicate their facilities thereon. In the design of transmission mains, the use of alternative routes is often possible, and location of utilities along these routes should also be obtained. In the City of Toronto and Metropolitan Toronto, and in other urban centres where there are Public Utility Co-Ordinating Committees, P.U.C.C. plans, if available, save considerable time, and therefore reduce route study costs. If there are no P.U.C.C. plans available, the design engineer then has to prepare a composite plan showing all the utilities, in order to determine where underground space is available for the installation of the water main. It is generally accepted in most areas that distribution mains be located approximately sixteen (16) feet from the street line, which places the main off the travelled portion of the roadway, usually in the boulevard or under a sidewalk, depending on the width of pavement. This is considered the ideal location, for excavated material may be used for backfill, whereas, if the main is located within the roadway, the roadway

authority will probably request granular material for backfill, and the cost of permanent pavement cut repairs would be greater than for a boulevard installation. In addition, one must consider the impact on traffic, inconvenience to the motoring public, private business, emergency vehicle, pedestrian traffic, maintenance, and access to the main for future connections. The problems involved in surveying and laying out a pipe line are affected by both the size of the main and its location. More detail and care are necessary as the size increases, or if the main is in an urban rather than a rural area.

The Public Utilities Act of the R.S.O. contains a section entitled "Prohibition as to laying main pipes and conduits within 6 feet of existing ones" as follows:

> Main pipes or conduits for carrying or conveying any public utility underground in any highway, lane or public communication shall not be laid by a municipal corporation or company within a distance of six (6) feet of the main pipes or conduits for carrying or conveying any public utility underground of any person without the consent of such person or the authority of the Ontario Municipal Board.

This section was undoubtedly written so that the owner of an existing utility would have some means of protection to prevent installation of new underground plant so close to existing utility as to endanger it or create a maintenance problem due to its proximity, and should be considered when selecting the alignment for a water main.

Once the composite plan or plans of the route or routes have been completed, and a tentative line selected for the main, and prior to any engineering survey work being performed, it is essential at this time to review the line or lines in the field. Note should be taken of embankments, ditches, water courses, railway crossings, and the size and location of all above-ground structures and trees on or adjacent to the proposed line. It may be necessary to change the proposed alignment to clear above-ground structures, and provide for easier and therefore less costly installation. If there are trees on the alignment, and there is no alternative location within the right-of-way, you can (1) save the trees by

tunnelling under or (2) remove the trees, which is probably the most economical method. If the latter method is chosen, you can expect a storm of protest from the residents along the route, and one should be prepared to replace the trees in a smaller size of the same species, which, in most cases, will appease most property owners.

Having considered the proposed location for the main in relation to existing utilities and practical above-ground difficulties, a complete topographical survey of the route should be carried out. If recent aerial photographs or P.U.C.C. sheets are available, considerable field work can be eliminated. The location of the main should be staked out in the field, and a profile over the centre of the main obtained. All sewer manholes should be entered and invert elevations taken; this is especially important where the water main is to cross over or under an existing sewer. In transmission water main design, sewer elevations parallel to the main must be checked to ensure that the main is placed at an elevation which will not cut off private storm and sanitary drains.

RAILWAY CROSSINGS

To avoid delay in obtaining railway pipeline crossing approvals, it is advisable to consult with railway Engineering officials prior to preparing plans.

If the main is to be installed in tunnel under the tracks, which is what most railway companies prefer, the following information should be obtained:

(1) Minimum acceptable cover over the tunnel at the tracks.

(2) Location of shafts in relation to tracks.

(3) Size, gauge and type of tunnel liner or steel jacking pipe.

(4) If work is to be done according to certain procedures and during certain hours.

(5) What railway supervision and inspection will be carried out during construction? Will the cost of this be absorbed by the owner or will the installation contractor be responsible for payment?

It should be noted that the railway companies prefer to deal with the owner rather than the contractor.

LAYOUT-PLAN AND PROFILE

Upon completion of the field work, a plan and profile, together with certain other details, are necessary for any water pipe line.

These should show:

(1) Horizontal and vertical distances, which is by survey station and elevation.

(2) Location of angles or bends, both horizontal and vertical.

(3) Degree of bends, degree or radius of curves, tangent distance for curves.

(4) Points of intersection with pipe centreline for tees, crosses and hydrants.

(5) Location of all line valves.

(6) Location of adjacent or interfering installations or structures.

(7) Location of existing utilities to be crossed over or under.

(8) Tie-ins with property lines, road or street centre lines, and other pertinent features necessary to define the right-of-way and locate pipe centreline clearly.

(9) Details of all specials, chambers, valve boxes, etc.

(10) Details of anchor blocks.

DEPTH OF COVER OVER MAIN

The minimum depth of cover over a water pipe should be determined by (1) The maximum depth of frost penetration in the locality where the pipe is to be laid.

Depth of maximum frost line plus a certain depth of unfrozen soil to act as a cushion is required. When a trench freezes, it expands approximately 12%, and in order that this expanding force is not transmitted through the pipe, a certain depth of "cushion" is necessary; (2) The direct effect of loading, both live and dead load, in addition to water hammer. The depth of cover and trench bottom are two primary factors in determination of pipe thickness. A cover of approximately 5 feet with a flat bottom trench and tamped or consolidated backfill usually results in the most favourable combination of earth and track hazard. The main should be installed on a three inch bed of sand or crushed stone on earth and six inches on rock, and the sand or stone thoroughly tamped so as to provide a uniform and continuous bearing and support for the pipe. Granular material should be placed up to twelve inches above the top of the pipe. The type of backfill material from twelve inches above pipe to surface will usually be determined by the location of the main within the right-of-way.

Failures in pipe lines usually result from settlement, careless installation, bedding on rocks, improper backfilling procedure, impact from loaded trucks, rupture from freezing because of insufficient cover, and improper or inadequate blocking.

THRUST BLOCKING

Good thrust-blocking practice is important in water main work and will assist in reducing problems such as breaks and leaks.

Wood plank, concrete blocks and poured concrete constitute the materials normally used for blocking pipe 12 inches and smaller. These blocking methods are generally effective. However, a check with maintenance departments indicate that leaks requiring emergency repairs often stem from two major sources; trench settlement and displacement of poorly blocked bends, fittings or plugs. These conditions will probably become more pronounced with the increased use of compression-type joints which are designed to slip home freely, and conversely, will slip free easily. All plugs, caps, tees and bends deflecting $22\frac{1}{2}^{\circ}$ or more should be provided with reaction blocking, or movement prevented by attaching suitable metal rod, clamps or harnesses. Further, the installation of the blocking should be carefully supervised because:

(1) The widespread use of compression fittings with slip joint characteristics as previously mentioned.

(2) Traffic, with its increased wheel load and frequency is causing greater vibration effect on fittings and blocking.

(3) Improved distribution systems may mean greater pressures even in the smaller mains.

(4) In cities where numerous utilities criss-cross the streets with their underground work, the bearing value of trench sides and bottoms have diminished. Thus when larger mains are extended in such streets, more conservative methods of anchorage should be used.

An understanding of the theory of thrust reactions and forces to be controlled will emphasize the need for blocking.

Forces exerted on dead ends are calculated by using the equation -

$$R = \frac{PA}{2000}$$

Where R = Force in tons
P = Pressure in pounds per sq. inch
A = Area of pipe cross-section in sq. inches

Forces exerted on reducers are calculated by using the equation -

$$R = \frac{PA_1}{2000} - \frac{PA_2}{2000}$$

Where R = Force in tons
P = Pressure in pounds per square inch
A_1 = Area of cross-section of the larger size of reducer
A_2 = Area of cross-section of the smaller size of the reducer in square inches

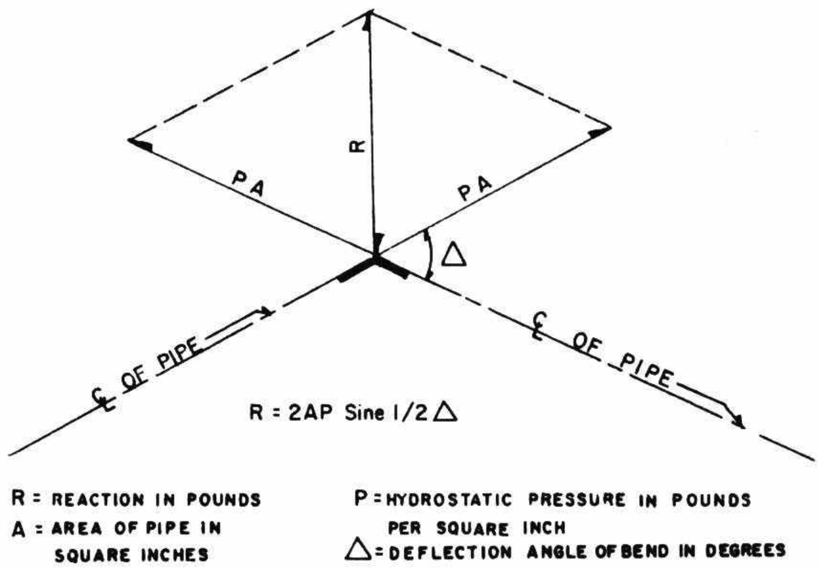


Fig. 1 - Hydrostatic thrusts at deflection of pipe lines.

Both dynamic and static thrusts come into play whenever there is deflection in a pipe line. In most cases the velocities in most mains are so low that dynamic thrust is insignificant. By contrast the static thrust is important and must be considered. Fig. 1 shows the force diagram and equation for determining the static thrust. The pressure should be the working pressure plus an allowance for water hammer. It is the function of the thrust block to transfer the loading of the unbalanced force from the fitting to the walls of the trench. Where the soil cannot carry the load, the thrust block must be of sufficient size and weight to anchor the fitting.

Soil	Minimum	Maximum
Rock, hard thick layers	200	-
Rock, equal good masonry	25	30
Rock, equal best brick	15	20
Rock, equal poor brick	5	10
Clay, always dry	4	6
Clay, fairly dry	2	4
Clay, soft	1	2
Gravel, coarse sand, firm	8	10
Sand, compact, firm	4	6
Sand, clean, dry	2	4
Quicksand, alluvial soil	½	1

Table I - Safe Bearing Value of Soil in tons per square foot

Table I, readily accessible in various texts, contains a tabulation of suggested bearing values for soils and rock formation; where trenches are shallow the weight of the undisturbed earth above the point of bearing may not be sufficient to prevent upward displacement of the soil and one should use a substantial reduction in these values.

The general practice to-day is to use poured-concrete thrust blocking for the following reasons:

(1) Concrete being plastic conforms itself to the shape of the fitting, giving complete and uniform support to the fitting.

(2) Modern ready-mix concrete assures uniformity of strength and is virtually indestructible.

(3) Unlike wood, concrete has great density. The added weight and friction resistance increases the support provided by bearing against the trench walls.

The following can also provide more effective blocking:

(1) Avoid unnecessary excavations at the sides of the trench.

(2) Excavate to provide a trench face that is as close to vertical as soil conditions allow.

(3) Careful consolidation of soil around the fitting to improve frictional resistance of pipe and fittings and to increase the weight of the soil over the pipe.

TYPICAL DESIGN CALCULATION USING FIG. I & TABLE I

(a) Calculate the thrust produced at a 90° bend in a 20-inch pipeline with the following conditions:

Working Pressure - 100 psi

Water Hammer Allowance - 100 psi

Static Force $R = \dfrac{PA \sin \theta/2}{1000}$

P = 200 psi (working pressure plus water hammer)

A = 314.16 square inches

$\theta = 90^\circ$

$$R = \frac{200 \times 314.16 \times 0.707}{1000} = 44.42 \text{ tons}$$

(b) Determine the block size required to resist the thrust if the soil is saturated silty-sand and is about average.

From Table I, bearing capacity for silty-sand is $\frac{1}{2}$ to 1 ton per sq. ft. Since the soil is average in that classification, use 0.75 tons per sq. ft.

From (a) Thrust = 44.42 tons

$$\therefore \frac{44.42}{.75} = 59.23 \text{ sq. ft. block size}$$

Check for block settling:

Assume block 8 ft. x 8 ft. (centre of block on centre line of pipe). For this pipeline centre line of pipe must be 5 ft. below ground. Assume block 2 ft. wide, then total block volume is 128 cu.ft. or 4.7 cu.yd. and would weigh 9.6 tons (concrete 150 lb./cu.ft.). Bottom of block is 8 ft. x 2 ft. Therefore loading = 0.60 tons per square foot. Block will not settle.

ANCHORAGE AND OVERHEAD PIPE LINES

Care must be exercised when overhead pipe lines have to be installed on hanger brackets on bridge structures. The nature of the method of support, etc., depends almost entirely on local conditions, service, etc. However, the following should be considered in the preliminary design work:

(1) Supports for pipe lines should prevent any movement in all directions except along the axis of the pipe, in order to allow for expansion or contraction which will be far greater than that for pipe buried in the ground.

(2) Hangers and supports should be designed and placed so that expansion in a pipe line takes place in every joint and not in one joint only. Dresser couplings are commonly used to allow for expansion and contraction.

(3) Loads affecting overhead pipe lines may be from wind loads, snow or ice loads and improper alignment or elevation of supports along the pipe line.

(4) Pipe lines when suspended above ground must be high enough to clear any trucks, etc. passing underneath, taking into consideration possible future conditions as well as present.

(5) As a structural beam, steel, cast or ductile iron pipe are reasonably strong. But, one must consider the weight of the pipe, and the water in it when determining the distance between supports. The limiting condition is usually the deflection of the pipe.

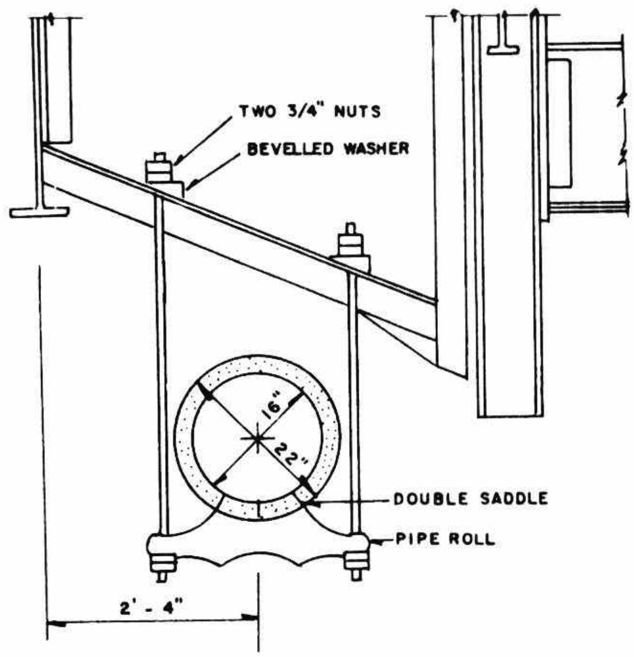


Fig. II -

Fig. II indicates a 16-inch diameter steel water main supported on hangers on a bridge structure, and the following an abstract from the section of the specification pertaining to the installation:

16" STEEL WATERMAIN

WORK

Under this item the Contractor shall supply and place 16" dia. .281" wall thickness steel watermain across the structure and behind the abutments to the limits indicated on drawing. Behind the abutments part of the main is cast iron. The interior surfaces of the steel pipe shall be cleaned, primed and given one coat of coal tar enamel and the exterior surface shall be cleaned, primed and coated with coal tar enamel having a bonded asbestos felt wrapper and finished with a single wrap of kraft paper. All such materials and application shall be in accordance with A.W.W.A. Specification C203.

The steel pipe snall be furnished with plain ends for style 38 Dresser couplings in accordance with A.W.W.A. steel water pipe specifications. Behind abutments the cast iron pipe and fittings shall be laid on a bed of consolidated sand at least three inches deep and the trench shall be filled with sand to a height of three inches above bells of the pipe joints. The price for 16" steel watermain shall include excavation, supply and laying the steel and cast iorn pipe, backfill and consolidation of excavation material and the provision of all couplings, bends, sleeves, reducers, distance pieces, concrete anchor blocks, testing the pipe line and supplying all equipment, tools, labour and all else necessary therefor, and all other work in connection therewith and incidental thereto.

16" PIPE BRACKETS

WORK

Under this item the Contractor shall provide and place hanger brackets for the 16" steel watermain. The $1\frac{1}{4}$" dia. hanger rods vary in length depending on location, they shall have at least 6" of thread at each end and all components shall be asphalt coated after insulation. Pipe saddles shall be Grinnell No. 187 for 16" pipe and 3" covering. Pipe rolls shall be Grinnell No. 171 for pipe size of 20", adjustable socket No.8-1.3/8". The price for 16" pipe brackets shall include the cost of drilling 4" x 4" x $\frac{1}{2}$" angles on the cantilever sidewalk brackets.

16" PIPE INSULATION

WORK

The steel pipe shall be insulated with Insulfab "M" blanket three (3) inches thick, 3.75 pounds per cubic foot, 24 inches in width in exact length to cover the circumference of the pipe, and diagonally wrapped with 45 pound felt both ways, with a 50% overlap.

All piping insulated as above on the bridge crossing shall be finished with .020 x 2S half-hard mill finished aluminum sheet. The sheeting shall be rolled to fit and held in place on the longitudinal seam only with stainless steel self-tapping screws on 3" centres. Alternatively, the sheeting shall be wired on with $\frac{1}{2}$" #16 B.W.G. monel metal bands, on 6" centres. The sheeting shall fit snugly over the insulation. All joint laps shall not be less than 3" and all laps shall be downward to shed water.

Where the section of the steel pipe is being installed in the ground the pipe line shall be insulated as above and in addition a one quarter ($\frac{1}{4}$) inch thick mastic coating shall be applied over the metal sheeting and over the ends of the insulation to protect the insulation from moisture.

No insulation shall be installed until after the pipe line has been chlorinated and pressure tested to the satisfaction of the Commissioner of Works.

MATERIALS

There is always a need for the exercise of engineering judgment in selecting the proper material and applying it in accordance with sound engineering principles, in order to effect an adequate design for a particular application.

The American Water Works Association has standards and handbooks pertaining to the manufacture, selection, design and installation of pipe and pipeline appurtenances, including valves and hydrants. These standards cover steel, concrete, cast iron, ductile iron and asbestos cement. Therefore, the designer of a pipeline has a wide choice. There are differences in design philosophy in the pipe industry on such matters as safety factor, water hammer allowance, corrosion and joint design. The person making a choice of type of pipe for a particular project must be aware of their differences in relation to the condition of his project. Field conditions may have an important bearing on the choice of pipe. These are generally more relevant on cross-country pipe lines. Transportation, terrain, soil conditions, backfill material, presence of rock, ground water conditions, soil corrosivity may affect choice of pipe material.

The risk of experimentation are too great when adequate materials of proven performance and long life are available.

VALVES

Water works valves can be classified in several ways. One method would be to differentiate between isolating and operating valves.

An isolating valve is a device for blocking off a section of pipe in a grid system or protecting operating valves on pumping units, so that inspection or maintenance can be carried out. An operating valve is one that is used frequently for starting, stopping, or regulating flow or for regulating pressure.

There are several types of valves used in water works service, the principal ones being gate, butterfly and check valves. There are also sluice gate, needle and plug valves which are designed for special applications. The following are used most often in water main installations.

Gate Valves

Gate valves are a favoured valve for distribution systems; they remain open most of the time and serve as isolation valves. Gate valves are either the solid-wedge or double-disc, parallel-seat type. The stems can be rising or non-rising, but only the non-rising stem valves are practical for burial in the ground. If a gate valve is intended for a throttling service, the solid wedge type, with close fitted guides, should be selected.

Butterfly Valves

The butterfly valves is coming into increasing use in water main installations. The valve possesses a great advantage in having only one moving part. The butterfly valve has advantages over other types of valves in its reduced size and short face-to-face installation length and ease of operation.

Check Valve

The primary purpose of a check valve is to prevent reversal of flow from one pressure district to another in a water main system.

The number and location of line valves depend on many factors. The spacing varies principally with the character of the territory traversed by the line. When an installation is in an urban area with connections to the local distribution system, one important consideration is the ability to sectionalize the line in order to maintain reasonable service in adjacent parts of the system. In the distribution grid, valves frequently may be only two blocks apart, while in large trunk mains they may be spaced a mile apart.

Installation of valves should be carefully supervised. If valves are rigidly installed in a pipe line and flanged joints are used, the whole assembly of pipe and valves is stressed by temperature changes, settlement and exceptional surface loads. To prevent a valve from being strained, there should be at least one flexible joint very close to it. This can be accomplished by installing a Dresser or Victaulic type coupling immediately adjacent to one of the flanges. Such a coupling makes installation and possible removal of the valve much easier.

CITY ENGINEERS ASSOCIATION DESIGN COURSE

WATER DISTRIBUTION SYSTEM

By: Mr. F. Booth

The problems we encounter every day are likely to be the same for each of our Municipalities, a gradual deterioration of the water system while, at the same time, the skyline fills up with high-risers. In this course, I would like to concentrate on the water distribution system as the main area of concern of the City Engineer and cover such aspects as pressure zones, water requirements, analysis of a pipe network, fire protection, selection of pipe sizes, and field investigations.

Purpose

Our purpose is to plan for the orderly development of the water distribution system to ensure that we can maintain the flow to the existing parts of the system and be able to extend the system for future development.

We should be able to find areas of potential weakness and give them special attention.

We should be able to plan ahead so that watermain construction can be co-ordinated with sewer and pavement construction.

Applied Principles

(1) The basis of design is to establish a design policy and have it approved by your Council. The design policy of the City of Toronto, in part, is as follows:-

A. Distribution System Criteria

The City shall, in general, provide adequate supply of water for domestic consumption during peak hour demand with a range in pressure between 100 and 30 p.s.i. at street level.

The City shall, in general, provide adequate flows for fire fighting purposes during a time of average domestic consumption on the day of maximum demand.

B. Domestic Distribution System

(i) Existing System

Many existing mains are upwards of 80 years old and their water carrying capacity is, on an average, 54% less than at the time of original installation. In general, they are structurally sound and shall be retained as the basic system.

The capacity of the existing mains shall be increased wherever possible by cleaning and relining. Where a capacity is required beyond the capability of this technique, the system shall be reinforced with new mains.

(ii) Future System Improvements

Where new mains are installed in high assessment areas with the existing main retained in service, consideration shall be given to locating the new main on the opposite side of the street to the existing main, thereby permitting new service connections to the closest main and minimizing cutting of pavement. Hydrants may be reconnected to the new main to take full advantage of its larger capacity.

Standard main sizes of 6, 8, 12, 16, 20 and 24 inches and larger, shall be adopted in order to minimize variations in size.

(iii) Mechanical Details

New watermains to be constructed in the City of diameter less than 16 inches shall be ductile iron, cement lined. The pipe thickness shall initially conform to class 3 for 6 inch; class 3 for 8 inch; class 4 for 12 inch and class 5 for 16 inch. Further study shall be given to the use of thinner walled ductile pipe and the use of concrete pipe for 16 inch diameter or larger.

All fittings shall be mechanical joint and consideration shall be given to the use of "Tyton" joint for ductile iron pipe.

Service connections shall be 3/4, 1-1/2 and 2 inch diameter of Type K soft copper tubing. Other permissible sizes are 4, 6, 8 and 12 inches or larger if required and of ductile iron.

Butterfly valves shall be in accordance with A.W.W.A. Spec. C-504, Class 150B. Gate valves shall be solid wedge, non-rising stem in accordance with A.W.W.A. Spec. C-500. All valves shall be constructed in valve chambers. Valve chambers shall be of concrete, either precast or cast in place.

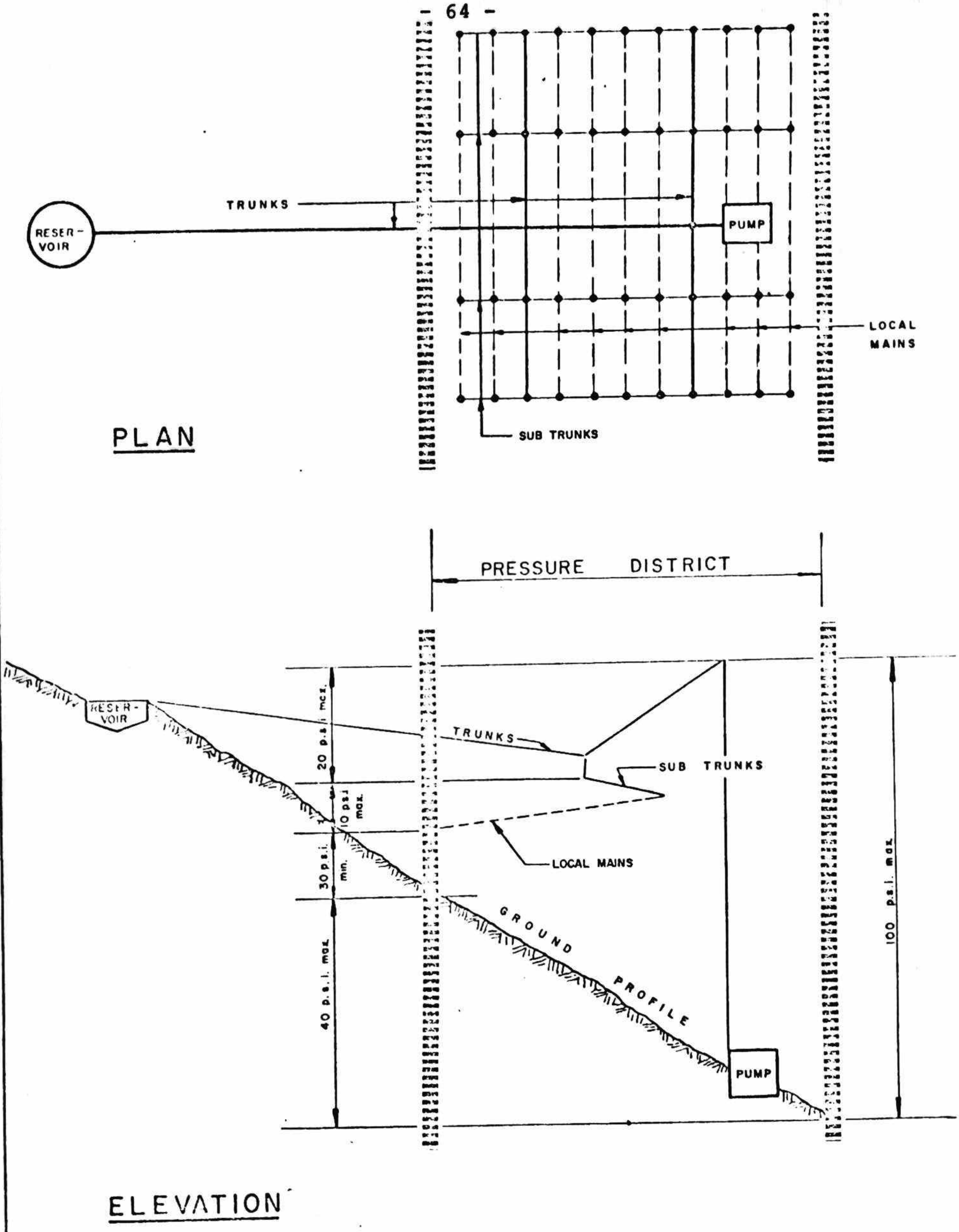
(iv) Hydrants

The City of Toronto hydrant design shall be continued. The advisability of adopting commercially available hydrants shall be studied including the merits of adopting the use of frost jackets to commercial type hydrants.

(v) Patterns

With the exception of hydrants, commercial valves and fittings shall be used in place of the City of Toronto design.

(2) The area to be supplied by water is marked-off by definite boundaries according to pressure. Ground levels should not vary by more than 100 feet (40 p.s.i.) and the pressure at a house on the highest ground should not be less than 30 p.s.i. (See typical water distribution system plan). If pressure loss between the pumps and the highest ground is 30 p.s.i. due to pipe friction, the initial pressure at the pumps would be 30+40+30 = 100 p.s.i. This is the maximum pressure that can safely be exerted on a pressure district.



TYPICAL WATER DISTRIBUTION SYSTEM

(2) continued:

Since we have allowed 30 p.s.i. for head losses, we can design our trunk mains for a head loss of 20 p.s.i. and 10 p.s.i. in sub-trunk and local mains.

(3) Sub-trunk principal mains are to be designed to carry the peak hour domestic flow requirements and local mains are to be designed for fire flow requirements.

Domestic Flow Requirements

The maximum day and peak hour rates of consumption are usually defined as multiples of average day rates. To obtain the present average day rate, we divide the annual consumption by the population and express the result as gallons per capita per day. This value would range from 30 g.p.c.d. to 100 g.p.c.d. Most systems are designed for a 100 g.p.c.d.

In residential districts, the peak demand usually occurs during the early evening, whereas in commercial and industrial areas the peak is experienced around mid-day and the early part of the afternoon.

The following peak hour to average day ratio for different land uses are realized in the City of Toronto:-

		Land Use	Ratio: Peak Hour / Avg. Day
1.	Evening Residential Peak Hour	Residential	3:50
		Apartment	2:00
		Comm. & Indus.	1:00
		Institutional	1:00
		Public	1:00
2.	Afternoon Commercial and Industrial Peak Hour	Residential	1:50
		Apartment	1:00
		Comm. & Indus.	2:00
		Institutional	3:50
		Public	2:00

Maximum day to average day ratio is 2.0 for design purposes.

Population will vary with land use and can vary from 30 to 300 persons per acre.

Fire Flow Requirements

The ability of a water system to serve consumers is determined by the amount of water delivered at a satisfactory pressure.

Water should be able to rise to the upper stories of buildings of moderate height and be able to satisfy the demands of the Fire Department.

Fire Flow Requirements (continued)

Generally, the following operating pressures are considered the desirable minimum:-

Residential Districts where buildings do not exceed 2-storeys.	40 p.s.i.
Apartments	60 p.s.i.
Industrial Areas	75 p.s.i.

The operating pressure at a hydrant for use by the Fire Department should not be less than 20 p.s.i.

Recommended rate of flow by the Fire Underwriters for fire protection is as follows:-

Single family residential	1,000 g.p.m.
High rise residential, Institutional, industrial, mercantile and other.	2,500 g.p.m.

Hydraulics

Some of the fundamental principles of hydraulics should be understood by all who are engaged in planning, design, or operation of distribution systems.

Measurement of pressure is accomplished by a Bourdon gauge graduated in pounds per square inch (p.s.i.) or in feet of head. One p.s.i. is equal to 2.31 ft. of water head or conversely 1 ft. of head is equal to 0.433 p.s.i.

Flow in Pipes

The volume of flowing water in a pipe is obtained from the Hazen-Williams formula provided the coefficient of roughness is carefully determined.

The formula is:

$$Q = 1.318\ C.A.R.^{0.63}\ S^{0.54}$$

Q = flow in cubic feet per second.

C = coefficient of roughness.

A = area of pipe in square feet.

R = hydraulic radius in feet.

S = slope of hydraulic gradient in feet per foot of pipe.

The quantity flowing in a given pipe will vary directly as the coefficient C.

Values of C for use in the H-W formula for cast iron are as follows:-

Pipe Size	New Pipe	Old - Unlined Pipe
6	100	50
8	110	60
12	120	65
16	130	80
20	140	80
24	140	80

Flow through an Orifice

To determine the amount of water flowing through a fire hydrant nozzle, or to obtain the amount of leakage through a hole in a pipe, the common orifice flow equation is used:-

$Q = CA\sqrt{2gh}$

Q = flow

A = Area of the opening

C = Coefficient of discharge

g = the acceleration due to gravity

h = head on the orifice.

Relating the Q under two different pressure heads, h_1 and h_2, we obtain the following equation:-

$$\frac{Q2}{Q1} = \sqrt{\frac{h2}{h1}}$$

Thus, if the pressure and flow are known for one set of values, we can calculate the flow for any other pressure condition.

Analysis of Pipe Network

(1) Manual Method

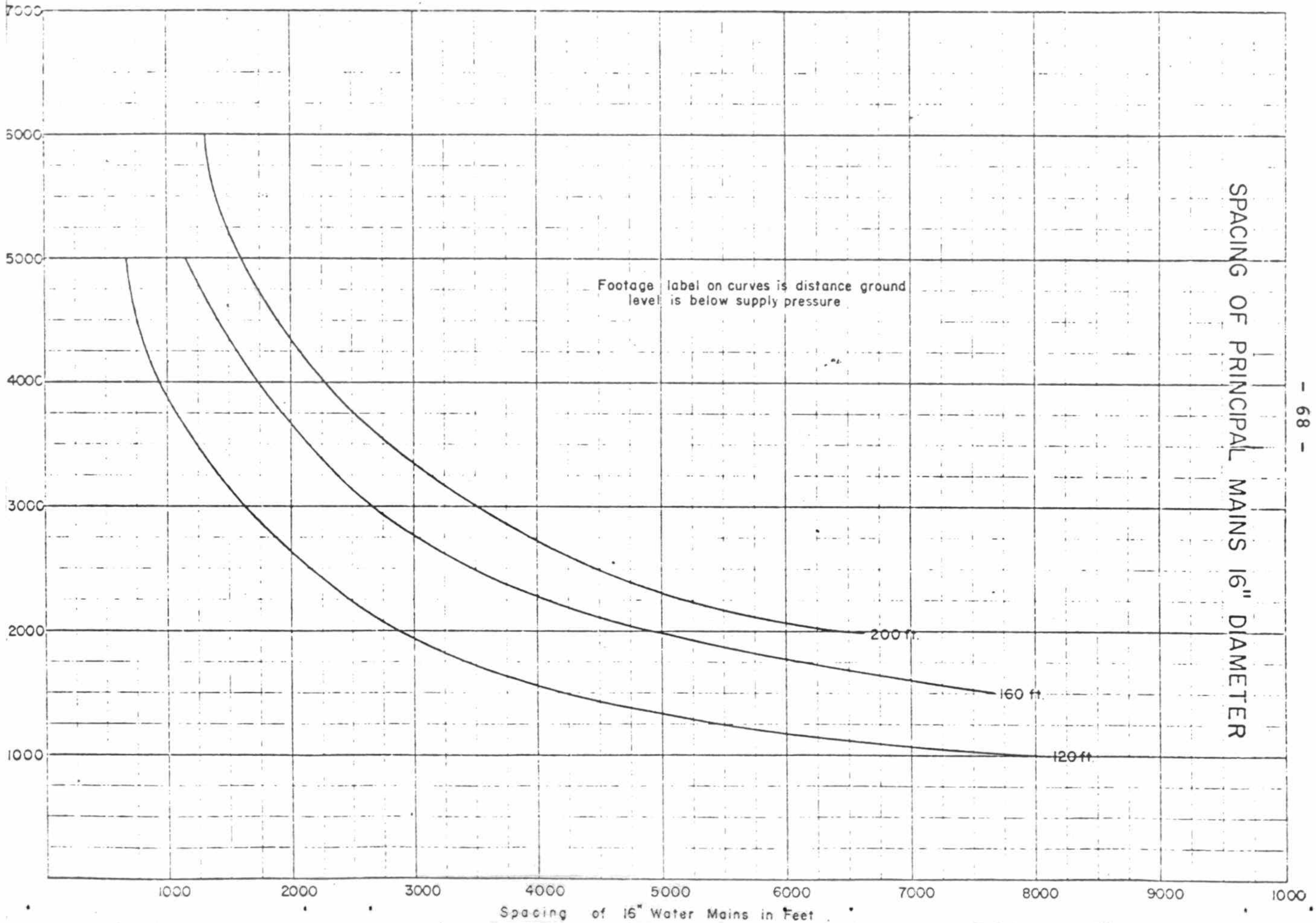
The spacing of principal watermains can be taken from curves shown on the accompanying pages. These curves were prepared from the information:-

Population - 300 ppa (gross area)

Consumption - 70 G.P.C.D.

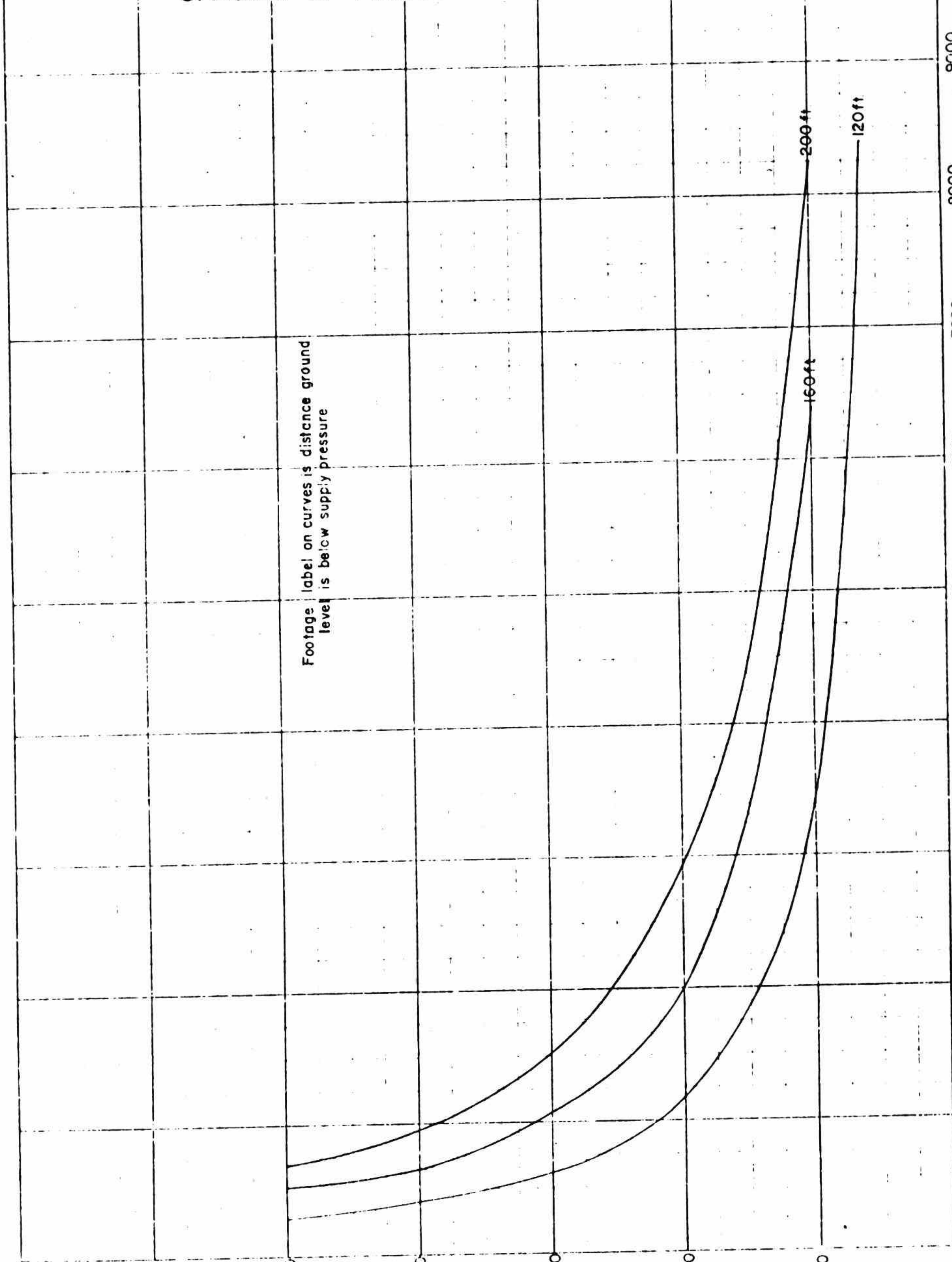
Peak Hour Factor - 3.0

$\therefore$ for one acre, the quantity of water to be supplied
= 300 x 70 x 3.0 = 63,000 gals. imp.



SPACING OF PRINCIPAL MAINS 16" DIAMETER

SPACING OF PRINCIPAL MAINS 12" DIAMETER



(1) Manual Method (continued)

The area which could be served by a principal watermain was calculated for head losses of 20', 60' and 100'.

Sample Calculations

Diameter of pipe = 12"

C = 120

Ground level = 120' below supply pressure.

Dist.to Trunk Main in Feet	Pressure of Trunk Main in Feet	Head Loss in Feet	Hydraulic Gradient Ft/1000	Cap. in M. G. D. x 1.5	Area Served in Acres	1000 Sq.Ft.	Pipe Spacing in Feet
500	100	20	40	7.35 Imp.	117.0	5100	10,2[illegible]

Note: The capacity of the main was adjusted by a factor of 1.5 to allow for numerous connections which, in effect, decreases the head loss along the main.

Diameter of Local Mains

Local mains must be capable of supplying water for a fire which may o[illegible] at the same time as the maximum day demand. This amount varies from [illegible] G.P.M. for low density residential buildings to 2500 G.P.M. for high-[illegible] apartments. For the ultimate plan, a flow of 2500 G.P.M. is required [illegible] be supplied in the middle of the block. There are three conditions of supply to be considered:-

1. The fire flow may be taken from hydrants on adjacent streets where the local mains are dead ended.

2. For blocks that are more than 600' apart, the fire flow would be taken from a single main fed from both ends of the street.

3. The fire flow may be taken from hydrants on adjacent streets where the local mains are fed from both ends of the street.

For conditions 1 and 2, the supply required is:

$$\frac{2500}{2} \times 1.2 \times 60 \times 24 = 2.16 \text{ M.G.D. (U.S.)}$$

For condition 3, the supply required is:

$$\frac{2500}{4} \times 1.2 \times 60 \times 24 = 1.08 \text{ M.G.D. (U.S.)}$$

The minimum pressure recommended by the Underwriters Association is 20 [illegible] allowing a loss of head in the local main of 100' - (20 x 2.31) = 54'.

The requirements for local watermains are related to the length of City blocks in the following table:-

CONDITION 1 2 supplies, both dead ended					
Fire Flow M.G.D.(U.S)	Dia. of Watermain	Hydraulic Gradient Ft/1000	Loss Feet	Length of Pipe Feet	Length of block supplied by Watermain
2.16	Old 6"	953	54	57	157
"	New 6"	263	"	205	305
"	New 8"	54	"	1,000	1,100
"	New 12"	6.5	"	8,300	8,400
CONDITION 2 2 supplies, fed from both ends					
2.16	Old 6"	953	54	57	214
"	New 6"	263	"	205	510
"	New 8"	54	"	1,000	2,100
"	New 12"	6.5	"	8,300	16,700
CONDITION 3 4 supplies, fed from both ends					
1.08	Old 6"	264	54	204	508
"	New 6"	73	"	740	1,580
"	New 8"	15.1	"	3,570	7,240
"	New 12"	1.79	"	30,000	60,100

(2) By Computer

The analysis of a pipe network is now done in our office with the aid of a computer employing the Hardy Cross Method of network analysis in which a flow is assumed for each pipe and then by trial and error the flow in the system is balanced.

Programmes are now available that can analyse an existing system or a proposed system and then design the network.

A manual of instructions for the user is part of each programme and preparing the necessary data is a matter of following a set of detailed instructions. The advantage of a computer programme is the complete results of the analysis are printed out for each pipe in the system. Illustration of a computer study will be shown.

The results of a computer analysis must be verified by actual field measurements before the computer model can be used for design purposes. This is done by comparing pressures taken at a group of hydrants with the calculated pressures.

Illustrative Problems

(1) Fire Flow Test

Find the amount of flow available for fire protection.

The number of hydrants may vary from 2 to 6. Static pressure at hydrant near centre of group is read, then a hydrant is opened and pressure read while hydrant is flowing.

Static pressures = 45 p.s.i. and 40 p.s.i.

Flowing pressure measured by pitot gauge = 23 lbs.

From table 1 flow = 597 gpm (Imperial).

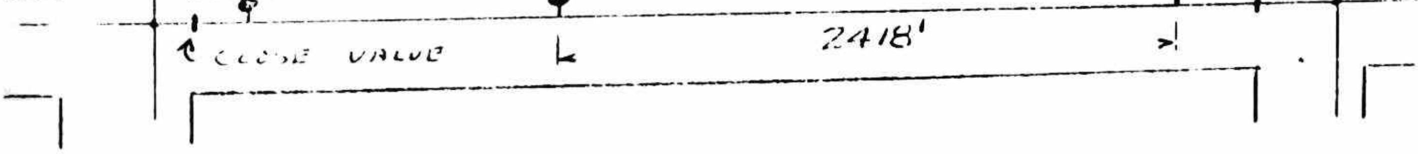
Calculate flow for a minimum pressure of 20 psi.

$$Q2 = Q1 \sqrt{\frac{h2}{h1}} \qquad h2 = 45-20 = 25 \text{ p.s.i.} \quad h1 = 45-40 = 5 \text{ p.s.i.}$$

$$Q2 = 597 \sqrt{\frac{25}{5}} = 1340 \text{ gpm.}$$

(2) Measurement of "C" - Roughness Coefficient

Three hydrants are located on a 6" watermain between two intersections.



(2) Measurement of "C" - Roughness Coefficient (continued)

Static pressures are measured at hydrants 1 and 2. Hydrant 3 is opened and flow measured and, at the same instant, pressure measured again at hydrants 1 and 2. Readings on gauges are:-

Hydrant 1 = $45\frac{1}{2}$ psi and 43 psi

Hydrant 2 = 49 psi and $13\frac{1}{2}$ psi

Hydrant 3 = $10\frac{1}{2}$ psi

from table 1, flow Q = 1.076 cfs;

head loss = difference between pressures taken at hydrants 1 and 2 = $(49-13\frac{1}{2}) - (45\frac{1}{2}-43)$

$= 33$ psi or 76.2 ft.

Hydraulic grade slope = $S = \frac{76.2}{2418} = .03153$ from the Hazen-Williams formula

$Q = 1.318CY^{0.63} S^{0.54}$ and rewriting and transposing, the equation becomes

$$C = \frac{Q}{S^{0.54}} \times 14.31$$

$$\therefore C = \frac{1.076}{.03153^{0.54}} \times 14.31$$

$$= \frac{1.076}{0.155} \times 14.31 = 99.2$$

Design of Watermain Extension

The Imported Foods Limited want to build on Mararoni Street and require a fire flow of 2500 gpm. The new watermain must be extended 450' and ground levels slope upwards 10 ft. Available pressure measured at nearest hydrant = 40 psi.

From the above information, the maximum allowable head loss is

40 psi - 20 psi - 10 psi x .433 = 15.67 psi.

In the Hazen-Williams tables, head loss is given in feet per thousand feet

$\therefore$ 15.67 psi x 2.31 = 36' per 450'

which becomes 80' per 1000'.

The new main should be designed for fire flow plus maximum day consumption.

No. of acres served = 5

Pop. = 300 ppa x 5 acres = 1500

Design of Watermain Extension (continued)

Max. day consumption = 1500 x 70 x 2 = 210,000 gals. or 0.21 MGD.

Total flow required = 2500 g.p.m. x 60 x 24 + 0.21 M.G.D. =

3.60 MGD + 0.21 M.G.D. = 3.81 M.G.D.

From Table 2, select a pipe diameter using cement lined pipe. C = 120

3.81 M.G.D. = 4.57 M.G.D. U.S.

A 12-inch diameter pipe is recommended.

TABLE 1

Outlet Pressure	Circular Outlet of 2½" with C = 0.9		C = 0.8		
p.s.i.	Gallons per Minute U.S.	Imp.	Gallons per Minute U.S.	Imp.	c.f.s.
.5 1	168	99 140	149	124	.332
2	235	198	209	176	.466
3	290	242	258	215	.575
4	336	280	299	249	.667
5	376	313	334	278	.745
6	412	343	366	305	.816
7	444	370	395	329	.881
8	475	396	422	352	.941
9	504	420	448	373	.999
10	530	442	471	393	1.050
11	557	464	495	412	1.104
12	583	485	518	431	1.155
13	606	505	539	449	1.202
14	629	524	559	466	1.247
15	650	542	578	482	1.289
16	672	560	597	498	1.331
17	692	577	615	513	1.371
18	713	594	634	528	1.414
19	732	610	651	542	1.452
20	751	626	668	557	1.490
21	770	642	685	571	1.528
22	788	657	701	584	1.563
23	805	671	716	597	1.597
24	823	686	732	610	1.632
25	840	700	747	622	1.666
26	857	714	762	635	1.699
27	872	727	775	646	1.728
28	889	741	790	659	1.762
29	905	754	805	670	1.795
30	920	767	818	682	1.824
31	935	779	831	693	1.853
32	950	792	845	704	1.884
33	965	804	858	715	1.913
34	979	816	870	725	1.940
35	994	828	884	736	1.971
36	1008	840	896	747	1.998
37	1022	852	909	757	2.027
38	1036	863	921	767	2.054
39	1049	874	933	777	2.081
40	1063	886	945	788	2.107
41	1075	896	956	797	2.132
42	1088	907	967	806	2.156
43	1102	918	980	816	2.185
44	1115	929	991	826	2.210
45	1127	939	1002	835	2.234

Orifice Formula -

$Q = CA \quad 2gh$

$C = .8 \quad D = 2\frac{1}{2}"$

$Q = 124.4 \quad P$ Imp. G.P.M.

1 c.f.s. = 374.4 Imp. G.P.M.
449.3 US G.P.M.

$C = .9 \quad D = 2\frac{1}{2}"$

$Q = 140.0 \quad P$ Imp. G.P.M.

TABLE 2

12-INCH PIPE.

A = 0.7854 D'

Discharge in: Gallons per 24 Hours.	Discharge in: Cubic Feet per Second.	Velocity in Feet per Second	Velocity Head, Feet.	Loss of Head in Feet per 1000 feet of length. (00) c = 140	(0) c = 130	(5) c = 120	(10) c = 110	(15) c = 100	(26) c = 90	(35) c = 80	Design Value c = 65
100,000	0.155	0.20	0.00	0.02	0.02	0.02	0.02	**0.03**	0.04	0.01	0.06
200,000	0.309	0.39	0.00	0.06	0.07	0.08	0.09	**0.11**	0.13	0.16	0.23
300,000	0.464	0.59	0.01	0.12	0.14	0.16	0.19	**0.22**	0.27	0.31	0.50
400,000	0.619	0.79	0.01	0.20	0.24	0.27	0.32	**0.38**	0.47	0.58	0.85
500,000	0.774	0.99	0.02	0.31	0.36	0.41	0.48	**0.58**	0.71	0.88	1.29
600,000	0.928	1.18	0.02	0.44	0.50	0.58	0.68	**0.81**	0.99	1.23	1.80
700,000	1.083	1.38	0.03	0.58	0.66	0.77	0.91	**1.08**	1.32	1.64	2.40
800,000	1.238	1.58	0.04	0.74	0.85	0.99	1.15	**1.38**	1.68	2.09	3.06
900,000	1.392	1.77	0.05	0.92	1.06	1.23	1.45	**1.72**	2.10	2.61	3.83
1,000,000	1.547	1.97	0.06	1.12	1.29	1.50	1.76	**2.10**	2.57	3.18	4.66
1,100,000	1.702	2.17	0.07	1.34	1.54	1.79	2.10	**2.50**	3.04	3.79	5.56
1,200,000	1.857	2.36	0.09	1.58	1.81	2.10	2.47	**2.94**	3.58	4.45	6.52
1,300,000	2.011	2.56	0.10	1.83	2.10	2.43	2.85	**3.40**	4.14	5.2	7.6
1,400,000	2.166	2.76	0.12	2.10	2.40	2.79	3.26	**3.90**	4.76	5.9	8.6
1,500,000	2.321	2.96	0.14	2.39	2.73	3.17	3.71	**4.43**	5.4	6.7	9.8
1,600,000	2.476	3.15	0.15	2.69	3.09	3.58	4.20	**5.0**	6.1	7.6	11.1
1,700,000	2.630	3.35	0.17	3.00	3.45	4.00	4.69	**5.6**	6.8	8.5	12.5
1,800,000	2.785	3.55	0.20	3.33	3.82	4.43	5.2	**6.2**	7.6	9.4	13.8
1,900,000	2.940	3.74	0.22	3.70	4.24	4.92	5.8	**6.9**	8.4	10.4	15.2
2,000,000	3.094	3.94	0.24	4.06	4.65	5.4	6.4	**7.6**	9.2	11.5	16.9
2,200,000	3.404	4.33	0.29	4.85	5.6	6.5	7.6	**9.0**	10.9	13.7	20.0
2,400,000	3.713	4.73	0.35	5.7	6.5	7.6	8.9	**10.5**	12.8	16.6	23.5
2,600,000	4.023	5.12	0.41	6.6	7.6	8.8	10.3	**12.3**	15.0	18.6	27.3
2,800,000	4.332	5.52	0.47	7.6	8.7	10.1	11.9	**14.1**	17.2	21.5	31.5
3,000,000	4.642	5.91	0.54	8.6	9.9	11.5	13.5	**16.0**	19.4	24.3	35.6
3,500,000	5.41	6.89	0.74	11.4	13.2	15.3	17.9	**21.3**	26.0	32.3	47.4
4,000,000	6.19	7.88	0.96	14.5	16.6	19.3	22.6	**27.0**	33.2	41.0	60.1
4,500,000	6.96	8.87	1.22	18.0	20.6	24.0	28.2	**33.6**	41.2	51	75.0
5,000,000	7.74	9.85	1.50	22.0	25.1	29.2	34.3	**41.0**	50.0	62	91.0
5,500,000	8.51	10.84	1.82	26.5	30.3	35.1	41.4	**49.4**	60	75	109.0
6,000,000	9.28	11.82	2.17	31.1	35.7	41.4	48.8	**58**	70	88	129.0
7,000,000	10.83	13.79	2.96	41.2	47.2	55	65	**77**	94	116	170.0
8,000,000	12.38	15.76	3.86	53	61	71	83	**99**	121	150	220.0
9,000,000	13.92	17.73	4.89	66	75	87	103	**122**	148	185	271.0
10,000,000	15.47	19.70	6.03	81	93	107	126	**150**	183	228	334.0

APPURTENANCES AND MATERIALS

IN

A WATER DISTRIBUTION SYSTEM

by

Mr. M. G. Bagshaw, P. Eng.

Paper prepared by

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LECTURE ON WATERMAIN APPURTENANCES

PART 1

PIPELINE ENGINEERING

Standards applicable to materials used in engineering works serve as an aid to, not a substitute for, engineering. There is always a need for the exercise of engineering judgement in selecting the proper material and applying it in accordance with sound engineering principles, in order to effect an adequate design suited to the needs of the pocketbook of the owner.

The purpose of standards is to consolidate a consensus of experience with, and acceptance of, a product at that point in its development and use when document-of-compliance criteria can be established for it. The standard must be compatible with a competitive manufacturing market and with the needs of users for obtaining a product physically suited to job needs, the product should be neither over- nor underdesigned. Cost and competitive position cannot be ignored.

AWWA Standards

The American Waterworks Association has standards and handbooks pertaining to the manufacture, selection, design, and installation of pipe and pipeline appurtenances, including valves and hydrants. These standards embrace steel, concrete, cast iron, ductile iron, and asbestos-cement pipe. The designer of pipelines has a wide choice. The various types of pipe are competitive with each other and the types are competitive even with themselves. There are differences in design philosophy among the competing groups of the pipe industry on such matters as safety factor, water hammer allowance, corrosion and corrosion protection, joint design, and pressure ratings - to name some of the most important. The person making a choice of type of pipe for his particular project must be aware of these differences and of their impact on the conditions of his project.

It would be wise for any pipeline engineering organization to keep abreast of the latest revisions in the AWWA Standards. An up-to-date list is available from the Association.

Pipe Materials

Change takes place slowly and with deliberation in the pipeline area of the water works industry. This is true for several very sound reasons. Pipe covered by AWWA Standards must have long life. Accepted depreciation practice give a range of 75-150 years. Experience records indicate that this range is conservative. Water supply is a stable industry. Obsolence is not the factor it is in manufacturing and other industries. Replacement is costly. The public nature of the water supply calls for sound investment. The risks of experimentation are too great when adequate materials of proven performance and long life are available.

In a review of the history of pipe for water system use, a pattern can be discerned. The Materials used reflect the technological limitations of their times. Wood pipe had a brief acceptance. Lead and ceramic pipe had some use. Cast iron was a natural entry on the scene. With the development of the steel industry, steel pipe and reinforced concrete pipe entered the picture. A dominant factor that stimulated progress was the desire to improve corrosion resistance. Unprotected ferrous metal pipes experienced serious loss in value because of reduced carrying capacity. Cement mortar linings were perfected and proved. Concrete pipe of many types which utilized the strength of steel and the corrosion resistance of concrete, appeared as a natural outgrowth of the development of structural reinforced concrete. Coal tar enamel coatings and linings for steel pipe were conceived and proven, making feasible the use of thin-walled steel pipe with its excellent strength and yieldability. This coating and lining material is in competition with cement mortar used for the same purpose. Asbestos-cement pipe came onto the scene when technology and quality control made possible the production of a product with consistent and predictable quality. It adequately needs the requirements for internal and external corrosion resistence. Ductile iron pipe is a recent entry. Its ductility permits it to yield under exceptional strains, offering great improvement over cast iron which fractures when subject to failure stress. Because of its greater strength and ductility, ductile iron pipe can be made thinner than similarly-rated cast iron pipe. This change will require greater concern where corrosion is a factor.

Homogeneous and reinforced plastic pipe are already on the scene. They appear to have great promise, but require experience before acceptance in metropolitan water systems. American Society for Testing and Materials standards are available for homogenous plastic pipe,and standards for reinforced plastic pipe are under consideration. Many types of plastic material are used for pipe. These materials have

different characteristics which must be understood. AWWA has a committee charged with the responsibility to develop a standard for the selection and design of plastic pipe for watermains and a manual for their installation. The committee is also charged with the task of developing similar documents for the use of plastic pipe for water service lines.

Selection of Pipe Type

The various types of pipe available have special applications where they are ideally suited. This fact generally restricts choice, depending on whether the line is a distribution, arterial, or transmission main. Distribution mains must be tapable for services at moderate cost. In small systems the same is true of arterial mains. Generally speaking, the traditional choice for distribution mains are cast iron and asbestos-cement pipe with ductile iron pipe rapidly gaining in desirability. In sizes above 12 inches, ductile iron pipe is often lower in cost than cast iron pipe; consequently, the use of cast iron for large diameter mains is declining.

Facility for thrust restraint inherent in the design of the pipe joint and limitations upon the use of thrust blocking may dictate a pipe choice. End thrust becomes an almost irresistable force for diameters above 30 inches and pressures above 100 psi. There are no standards for joint restraint, and there are many types in use with different types of pipe. Inherently they restrict the deflectability of the joint. The most conservative design may be a flexible pipe with a rigid joint (as, for example, in a welded steel pipeline), but there then may be limitations on type of lining to use, particularly where the pipe is too small for a man to enter to coat effectively the welded portion. Cement lining in place can be used effectively in welded steel lines.

Location Conditions

Field conditions may have an important bearing on the choice of pipe. These are generally more relevant in crosscountry pipe lines. Transportation, terrain, soil conditions, backfill material, presence of rock, ground water conditions, soil corrosivity, and available labour skills may affect choice of pipe material.

Design Pressure

The use of pressure-class ratings for pipe probably goes back to the 1908 AWWA standard for cast iron pipe, where alphabetical class designation were given in 100 foot head increments, beginning with "A" as the lowest for a 100 foot head. Later these were changed to pressure-designated classes in 50 psi increments, beginning with Class 50 and ending with Class 350. More recently, cast iron pipe ratings were again changed. The cast iron and ductile iron pipe standards now use a thickness class method and give rated working pressures for different laying conditions and depths of cover. The asbestos-cement pipe standards use pressure ratings for three classes, 100, 150, and 200, and these too are are tied in with selection curves that relate working pressure, laying conditions, and depth of cover. There are AWWA standards for the strength and thickness design of cast iron and ductile iron pipe and for the selection of asbestos-cement pipe based upon accepted engineering principles for the consideration of external and internal loading on pipe.

With regard to the selection of steel and concrete pipe the user does not have available the convenient rating references applicable to the types of pipe previously mentioned. For the users of steel pipe, however, the comprehensive Steel Pipe Manual mentioned is published by AWWA and is available. Another AWWA Manual, Installation of Concrete Pipe, is not so comprehensive and does not cover pipe design.

In the use of the steel pipe standards, the user must specify the design pressure, the standards designation and grade of steel required, the inside or outside diameter, and the working pressure. He must make a decision as to the need for a corrosion allowance when he gives the thickness and must make an all important determination as to the amount of water hammer allowance, if any, to include in the designated working pressure. It is general practice in the water works industry to use design steel stresses equal to 50 percent of the yield strength. The user must be familiar with this practice, but also must decide whether he wishes to allow steel stresses to rise up to the yield point in resisting surge or water hammer pressures. He also must be able to determine the amount of surge pressure to be expected in his system.

In the use of the concrete pipe standards, the user must establish the design pressure, the external loading, and be able to understand the basis of design of the pipe. He must evaluate the probable surge pressure and decide if it should be included in the working pressure or be permitted to encroach on the stress capability up to the yield point of the steel.

External Loading

The external loading conditions providing the basis for the selection of various pressure rated cast iron and ductile iron pipe are set forth in standards for thickness design of cast-iron pipe and in the parallel standard for the design of ductile iron pipe. The external loading conditions for the selection of pressure rated asbestos-cement pipe are also given in an AWWA Handbook. There are points of difference and it is appropriate that users of these pipes study and understand the differences. The judgement and experience of the industry injected into these design standards relate especially to the matter of safety factor, truck load, and need for water hammer allowance.

It is difficult to distinguish between underdesign and overdesign when comparing types of materials or when evaluating the degree of conservatisim in design. It is significant that the bases of design are only concerned with ring stress under load. Beam strength in relation to a rational basis of design is not evaluated. Many failures, especially in small diameter cast iron and asbestos-cement pipe are known to be due to beam stresses because of unequal settlement or partial support on unyielding soil or rock.

With regard to steel and concrete pipe, the user must establish the loading and laying conditions and the combinations thereof which he considers critical to the application of the pipe to his particular job. This requires a through knowledge of pipeline engineering and pipe design. Where the user is considering competing types of pipe, he must evaluate points of difference as they pertain to his application. Such matters as safety factor, effectiveness of protective coatings, and pipe deflection in the trench for the method of backfilling and the field load conditions deserve special scrutiny.

Corrosion Considerations

Internal corrosion with the accompanying loss in carrying capacity cannot be tolerated. Most old cities are paying dearly to restore the carrying capacity of old systems. Properly designed and applied cement mortar linings are providing effective protection in ferrous metal pipes today. For large diameter pipe, coal tar enamel linings are effective. AWWA standards are available for shop applied cement mortar and coal tar enamel linings. Cement mortar lining of old and new pipe in place is also covered by an AWWA standard. Asbestos-cement and concrete pipe provide the required corrosion resistance to maintain the strength and carrying capacity of the pipe line. In distribution system laterals, the control of free lime in the cement, by choice of cement and additives and by proper curing methods is desirable because of free lime's effect on the mineral content and taste of water. Asbestos-cement pipe can be manufactured with minimum free lime to avoid this problem. The bituminous seal coat used in the factory applied cement linings of cast and ductile iron pipes effectively solve this problem.

When using large diameter flexible pipes such as steel, ductile iron with cement linings and to a lesser extent with coal tar enamel linings, and pretensioned steel cylinder concrete pipe, attention must be given to limiting deflection due to external loadings. Backfilling and bedding methods are available to limit the deflection. It is most critical during construction before the line is under pressure. Good design requires that deflection (expressed as a percentage of the original diameter) be limited to two percent. Coal tar enamel linings can tolerate more deflection; however, control during installation cannot be relaxed. In large diameter installation, the use of stulls to elongate the vertical axis is effective in assuring a round pipe after construction is completed.

The proponents of prestressed concrete pipe consider it essential that the concrete lining be under compression to avoid hair cracks in the lining which would allow water to come in contact with the steel of the cylinder. In steel pipe and nonprestressed reinforced concrete pipe, hairline cracks are avoided by including the water hammer or surge pressure in the design pressure and by limiting steel stresses to a value of one half the yield point. The pipe user is confronted with making a decision on these points for which the industry provides no assured answer. It may be that the choice lies in selecting the degree of conservatism.

External corrosion must be avoided. Generally there is some experience in the area. Where soils are suspect, soil corrosivity tests should be made and special consideration given to the protection of ferrous metals. Cast iron pipe has a good experience record in most soil environments. Ductile iron has been shown to have similar properties; however, since it is cast to thinner sections, more attention should be given to the actual soil conditions where it is used. Where soils are high in organic matter and soil moisture, or where the water table is above the pipe, corrosion can be expected.

The use of a loose fitting plastic film tube is advocated for special corrosion protection of cast iron and ductile iron pipe. Cathodic protection is sometimes indicated. It should be used as an adjunct to a good protective coating. In large pipeline installations,where the line is a continuous conductor, the use of buried magnesium and anodes at regular intervals along the line is sometimes good insurance. The problem must be faced realistically where the pipeline is a continuous conductor, as in the case of a steel line with welded joints. Where high resistance joints (rubber ring joints) are used, the designer is faced with the vexing problem of whether to bond or not to bond. A high resistance pipeline is desirable where soil currents are not present. The application of cathodic protection to such a pipeline is impractical at a later date. General practice in the water works field is not to bond unless severe corrosive conditions are anticipated.

The widespread use of granular backfill gives opportunity to reduce corrosion potential when sand is used for backfill and especially when limestone screenings can be used. To be fully effective, the granular backfill should extend completely under the pipe to a minimum depth of 3 inches.

Cost Considerations

If the user of pipe wishes to make a selection of a type of pipe from all possible choices, he must exercise judgement and understand pipeline construction. He must be familiar with the points of difference between the design of the various types of pipe and with their shortcomings and limitations. If he chooses a type of pipe which requires special handling and backfilling, he must be prepared to provide it; and he must evaluate that whole cost in making a choice. In the preparation of plans and specifications for large pipelines, accepting bids on alternate types of pipe may be good business. The engineer should be able to make a clear and logical choice when bids are in.

Basically what is being purchased in a pipeline is carrying capacity and trouble-free, longlife service. If the purchaser is concerned about the amount of carrying capacity he purchases for his pipe dollar - and he should be - he should evaluate the carrying capacity of competing types of pipe, because they do vary. Inside diameters are not the nominal diameter. There may be considerable differences in the different types of pipe due the space occupied by different types of linings. For example the inside diameters of similarly rated cast iron, ductile iron and asbestos-cement pipe for nominal diameter of 8 inches is 8.01, 8.26 and 7.85 inches respectively.

The thickness of cement linings in cast and ductile iron pipes is very thin, 1/16 inch for sizes 3-12 inches, 3/32 inch for sizes 14-24 inches, and 1/8 inch for larger sizes. Cement linings used in steel pipe are thicker. In the size ranges most commonly used, they are: 3/8 inch for sizes 24-36 inches, and ½ inch for larger sizes. Coal tar enamel linings are 3/32 of an inch thick.

Roughness factors being equal, the carrying capacity of pipe varies directly with the 2.63 power of the inside diameter, based on the Hazen Williams formula. Thus, a 1 per cent difference in diameter produces a 2.6 percent difference in carrying capacity.

The achievement of long life is as much a matter of engineering expertise in the design and construction of a pipeline as in the choice of pipe. Paramount is the selection of the proper size so that obsolescence or premature replacement does not take place. This is largely a matter of economics. Thorough supervision and inspection of the installation and, where in doubt, of the manufacturing process are essential to the attainment of a long lived maintenance free pipeline.

Joint Restraint

One of the major problems in the design of large diameter pipelines is the method of joint restraint to be used. Failure to design restraint capability properly into the line may well be responsible for most of the failures experienced. There seems to be no clear-cut answer to this problem. The forces encountered are enormous. For example, a 150 psi pressure results in a thrust of approximately 24 tons for 20 inch pipe and 210 tons for 60 inch pipe. Surge pressures can add materially to these values. The longitudinal stress due to end thrust, if exterior blocking is not used, is one-half the hoop stress and must be adequately provided for in the barrel of the pipe and in the type of joint restraint used. The effect of these stresses in combination with other stresses, such as bending stresses due to external loads or nonuniform bedding, must be considered.

In large diameter pipeline installations, the use of concrete blocking is not the best practice in city streets. There is always the danger that an adjacent excavation will remove the restraining earth behind the blocking. Good design calls for a restraint of joints for a distance in either direction from the fitting or appurtenance which is responsible for the thrust sufficient to transfer the stress to the soil through friction until the longitudinal stress is reduced to zero. The determination of that distance requires application of the principles of soil mechanics.

The development of adequate joint restraint in pipe has been in a state of flux. More attention has been given to this problem in recent years. There are few standards and some of those have shortcomings. If external clampsand tie rods are used, adequate corrosion protection should be provided in order for the line to have no weak links. Many failures have been caused by corrosion of tie rods and clamping divices.

Conclusion

Standards are a valuable tool in pipeline engineering, but the design of an adequate, long-lived, maintenance free pipeline requires the application of sound engineering in design, choice of pipeline materials and installation.

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PART 2

THICKNESS DESIGN OF DUCTILE-IRON PIPE

The American Water Works Association has published a standard for the thickness design of Ductile Iron pipe. This publication has the AWWA nomenclature of H3-65 and also the American Standards Association nomenclature of A21.50-1965.

The method of thickness design of ductile iron pipe presented in the Standard is based on flexible pipe principles developed at Iowa State College by M. G. Spangler and his Associates. The principal characteristics that distinguish flexible pipe from more rigid types of pipe are as follows:

1. In a flexible pipe the bending stress from trench load is reduced by the lateral soil reaction that is developed as the pipe deflects under the trench load and pushes outward against the sidefill soil.

2. A flexible pipe, initially deflected by trench load, is partially rerounded by internal pressure, and the bending stress of trench load is thus reduced.

3. A flexible pipe is usually required to carry less earth load than a more rigid pipe, because the flexible pipe, in deflecting under the earth load, transfers a significant part of the load to the sidefill soil columns.

These characteristics were expressed mathematically by Spangler in equations from which may be calculated the earth loads on flexible pipe and the bending stresses and deflections of flexible pipe when subjected to: (1) external trench load and no internal pressure and (2) external trench load in combination with internal pressure. These equations are applicable to pipe made of various elastic metals, of which ductile iron is one.

The Standard details design criteria and factors used. Some of the more interesting factors are as follows:

1. Numerous calculations have shown that the deflection of pipe designed for bending stress according to the procedure in the Standard will not exceed 2% of the outside diameter of the pipe, a deflection at which the cement linings will not be damaged.

2. The Standard is based on laying the pipe in a flat bottom trench under two backfill conditions. One condition, A, is for untamped backfill while the other condition, B, is for tamped backfill.

3. An allowance of 100 psi is added for water hammer surges to the design working pressure.

4. The Standard is based on a unit weight of 120 pounds per cubic foot for backfill soil. Soil weights generally vary from 110-130 pounds per cubic foot. Experience has shown that 120 pounds per cubic foot is commonly considered to be a conservative value for most installations.

5. The truck super load allowance in the Standard is based on two passing trucks with adjacent wheels 3 feet apart, 9,000 pound wheel load, unpaved road or flexible pavement, and 1.50 impact factor. These loads, in most cases, equal or exceed the static load from a single AASHO H-20 truck with 16,000 pounds on each rear wheel. These truck super-loads are based on having the design depth of cover over the pipe. Consideration should be given to the loads that may be transmitted to the pipe if either truck super loads or heavy construction equipment is permitted to pass over the pipe at less than the design depth of cover.

6. Comparative corrosion tests of cast iron and ductile iron pipe over a 14 year period in three corrosive soils proved that the corrosion resistance of ductile iron pipe is essentially the same as that of gray cast-iron pipe. Therefore, the design thickness is increased by 0.08 inches to provide an arbitrary allowance for external corrosion. Present knowledge indicates that this corrosion allowance is adequate for most soils.

7. The standard thickness includes a casting tolerance, and the standard weight is calculated from this thickness. The average thickness of a pipe is controlled by the minimum weight limitation. The minimum thickness is limited by the casting tolerance.

8. Some utilities have established minimum numbers of threads for tapped holes in pipe. Consideration should be given to pipe wall thickness and tap size to insure serviceable threaded connections. Service conditions should indicate the extent of full thread engagement necessary and the necessity for use of outside sealing corporation stops, tapping saddles, or other fixtures. To facilitate checking the number of threads for taps of various sizes in different pipe thicknesses, appropriate tables are provided in the standards for ductile iron pipe.

Design

The required thickness of ductile iron pipe is determined by considering trench load and internal pressure separately. Calculations are made for the thickness required to resist the bending stress of trench load and the thickness required to resist the hoop stress of internal pressure; the larger of these is selected as the net design thickness. To this net thickness is added a corrosion allowance to obtain minimum manufacturing thickness and a casting tolerance to obtain the total calculated thickness. Finally, the thickness for specifying and ordering is selected from a table of standard class thicknesses.

The following is the procedure for selecting thickness.

Step 1 - Design for trench load: The net thickness required for trench load under the more usual field conditions (Laying Conditions A and B) may be readily determined using tables 1 and 2 of the Standard. These tables are reproduced as part of these notes. The procedure is as follows:

a) From Table 1 determine W , the trench load per linear foot of pipe, which is earth load (W_e) plus truck super load (W_t) as applicable.

b) Convert the above trenchload to pounds per linear foot per inch diameter by dividing W_c by the outside diameter in inches(D) from Table 1 $\left(\frac{W_c}{D}\right)$.

c) With the above value of $\frac{W_c}{D}$ and the appropriate laying condition (A or B), determine that diameter-thickness ratio, $\frac{D}{t}$ from Table 2.

d) Determine the required net thickness of the pipe wall in inches, t, by dividing the outside diameter, D, by the diameter thickness ratio, D/t . This will be the net thickness of the pipe wall required to support the trench load under the conditions assumed. It should be noted that the outside diameter, D, is constant for all thicknesses of pipe of the same nominal size. Its substitution in place of the theoretically correct mean diameter or inside diameter, as applicable, introduces no significant error.

Step 2 - Design for internal pressure: The net thickness required for internal pressure is computed by the equation for hoop stress:

$$t = \frac{pD}{2s}$$

in which t is net thickness in inches, p is working pressure plus surge pressure, in pounds per square inch, D is outside diameter of pipe in inches, and s is design hoop stress (16,800 psi).

An allowance of 100 psi surge pressure should be made. When surges more than 100 psi are anticipated an analysis of the conditions should be made.

Step 3 - Selection of Net Thickness and Addition of Allowances:

a) The net thickness, t, is selected from Step 1 or Step 2, which ever thickness is larger.

b) A corrosion allowance of 0.08 inches is added to the net thickness, t. The resulting thickness is the minimum manufacturing thickness, t_1. Where severe corrosion is anticipated, an analysis of the condition should be made.

c) A casting tolerance is added to the minimum manufacturing thickness, t_1, and the resulting thickness is the total calculated thickness.

The following table indicated the casting tolerances:

Size, inches	Casting Tolerances, inches
3-8	0.05
10-12	0.06
14-42	0.07
48	0.08

Step 4 - Selection of Standard Thickness: In specifying and ordering pipe, the total calculated thickness from Step 3 is used to select one of the standard class thicknesses in Table 6 of the Standard (which is reproduced herein). The standard thickness nearest to the calculated thickness is selected. When the calculated thickness is halfway between two standard thicknesses, the larger of these is selected. When the calculated thickness is less than the smallest standard thickness in Table 6, the smallest standard thickness is selected.

Design Example for Selecting Thickness for 24-inch Pipe:

In order to familiarize you with the tables a design example is used. The problem is to calculate the thickness for 24-inch diameter ductile iron pipe laid on flat bottom trench with tamped backfill under 5 feet of cover for working pressure 150 psi.

Step 1 - Design for Trench Load:

Earth load from Table 1 = 1,290 lb/lin. ft.
Truck Load from Table 1 = 769 lb/lin. ft.

Trench load, W_c = 2,059 lb/lin. ft.

Divide trench load by outside diameter of pipe:

$$\frac{W_c}{D} = \frac{2{,}059}{25.80} = 80 \text{ lb/ft/in.}$$

Find $\frac{W_c}{D}$ = 80 in Table for Laying Condition B, and read the required value of the diameter-thickness ratio, $\frac{D}{t}$ = 112.

Divide this ratio into the outside diameter of the pipe to obtain net thickness:

$$t = \frac{25.80}{112} = 0.23 \text{ inches.}$$

Step 2 - Design for Internal Pressure:

a) Water Pipe:

Working Pressure = 150 psi
Surge pressure = 100 psi

Design pressure = 250 psi.

Then,

$$t = \frac{250 \times 25.80}{2 \times 16{,}800} = 0.19 \text{ in.}$$

Step 3 - Selection of net thickness and addition of allowances:

The larger of the thicknesses is given by the design for trench load, Step 1, and therefore 0.23 in. is selected as the net thickness for water.

Step 3 - Continued

Net thickness	= 0.23 in.
Corrosion Allowance	= 0.08 in.
Minimum manufacturing thickness	= 0.31 in.
Casting tolerance	= 0.07 in.
Total calculated thickness	= 0.38 in.

Step 4 - Selection of standard thickness: The total calculated thickness of 0.38 in. is less than the smallest standard thickness shown in Table 6. Therefore, the smallest standard thickness, 0.41 inches, Class 1, is selected for specifying and ordering.

Conclusion

The American Standard for the thickness design of Ductile Iron Pipe is a handy tool for use if the user is aware of its limitations. The Standard should not be used without familiarity with the design criteria used in developing it.

Thickness Design of Cast Iron Pipe: AWWA also has a Standard (H1-67) for the thickness design of cast iron pipe. This standard is set up in a similar basis to the one previously discussed for ductile iron pipe. In addition, the Standard provides for the thickness determination for pipe on piers or piling above ground or underground which may be of interest to designers.

Furthermore, this Standard discusses in detail unusual installation conditions with respect to earth loads. The ditch condition is the most common method of installing cast iron pipe and is the basis of the earth load shown in the Tables in the Standard. Ditch condition denotes pipe laid in relatively narrow trench and backfilled to the original ground surface. The trench width at the top of the pipe determines the load and the ditch may be widened above the top of the pipe for installation convenience without increasing the load on the pipe.

TABLE 1

Earth Loads (W_e) *and Truck Superloads* (W_t)

Size in.	Outside Diameter (D) in.	Earth Load	Truck Load	Earth Load	Truck Load	Earth Load	Truck Load	Earth Load	Truck Load
		lb/lin ft							
		2.5-ft Cover		3.5-ft Cover		5-ft Cover		8-ft Cover	
3	3.96	182	162	260	81	376	54	609	40
4	4.80	226	297	324	162	471	81	765	54
6	6.90	309	567	448	324	657	189	1,075	94
8	9.05	380	783	557	486	824	297	1,356	148
10	11.10	402	972	582	621	854	378	1,397	189
12	13.20	423	1,161	607	756	884	459	1,438	243
14	15.30	445	1,217	633	807	915	540	1,478	270
16	17.40	466	1,307	658	879	945	590	1,519	324
18	19.50	488	1,400	683	964	975	632	1,560	364
20	21.60	540	1,524	756	1,076	1,080	729	1,728	410
24	25.80	645	1,662	903	1,159	1,290	769	2,064	462
30	32.00	800	1,925	1,120	1,356	1,600	918	2,560	564
36	38.30	958	2,182	1,340	1,577	1,915	1,090	3,064	632
42	44.50	1,112	2,354	1,558	1,750	2,225	1,239	3,560	717
48	50.80	1,270	2,527	1,778	1,922	2,540	1,400	4,064	814
		12-ft Cover		16-ft Cover		20-ft Cover		24-ft Cover	
3	3.96	920	27	1,230	14	1,538	9	1,852	7
4	4.80	1,157	40	1,549	27	1,942	18	2,334	14
6	6.90	1,632	68	2,188	40	2,524	26	2,647	20
8	9.05	2,066	94	2,592	54	2,815	36	2,965	28
10	11.10	2,121	108	2,698	68	3,032	45	3,308	35
12	13.20	2,176	122	2,803	81	3,249	53	3,651	41
14	15.30	2,230	135	2,909	94	3,466	62	3,994	48
16	17.40	2,285	162	3,014	122	3,683	81	4,337	62
18	19.50	2,340	189	3,120	135	3,900	89	4,680	69
20	21.60	2,592	216	3,456	162	4,320	107	5,184	83
24	25.80	3,096	256	4,128	176	5,160	116	6,192	90
30	32.00	3,840	324	5,120	216	6,400	143	7,680	110
36	38.30	4,596	378	6,128	256	7,660	169	9,192	131
42	44.50	5,340	459	7,120	310	8,900	205	10,680	158
48	50.80	6,096	513	8,128	351	10,160	232	12,192	179

SOURCE: American Standard for Thickness Design of Ductile Iron Pipe (A21.50-1965).

There are two unusual installation conditions that are also discussed. These are positive projection and negative projection embankment conditions. Positive projection condition denotes pipe laid on top of a subgrade and covered with fill. Negative projection condition denotes pipe laid in a trench in the subgrade and covered with fill which extends substantially above the subgrade. The methods for calculating earth loads for these two unusual conditions are set out in the Standard.

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TABLE 2

Diameter-Thickness Ratios

$\frac{W_e}{D}$ lb/ft/in. A*	$\frac{W_e}{D}$ lb/ft/in. B†	$\frac{D}{t}$‡	$\frac{W_e}{D}$ lb/ft/in. A*	$\frac{W_e}{D}$ lb/ft/in. B†	$\frac{D}{t}$‡	$\frac{W_e}{D}$ lb/ft/in. A*	$\frac{W_e}{D}$ lb/ft/in. B†	$\frac{D}{t}$‡
62	73	120						
62	74	119	94	108	89	189	200	59
63	74	118	96	109	88	194	206	58
64	75	117	98	111	87	200	212	57
65	76	116	99	113	86	207	218	56
66	77	115	101	115	85	214	225	55
66	78	114	103	117	84	221	232	54
67	79	113	105	118	83	230	240	53
68	80	112	107	120	82	238	248	52
69	81	111	109	122	81	246	257	51
70	82	110	111	124	80	256	265	50
71	83	109	114	127	79	265	275	49
71	84	108	116	129	78	276	285	48
72	85	107	119	132	77	287	297	47
73	86	106	122	134	76	300	309	46
74	87	105	124	137	75	312	321	45
75	88	104	127	140	74	326	335	44
76	89	103	130	142	73	340	349	43
77	90	102	133	145	72	357	365	42
78	91	101	136	149	71	373	382	41
80	92	100	139	152	70	391	400	40
81	93	99	142	155	69	411	420	39
82	94	98	146	159	68	432	441	38
83	96	97	151	163	67	456	464	37
84	97	96	155	166	66	481	489	36
86	99	95	159	170	65	508	516	35
87	100	94	163	175	64	538	545	34
88	102	93	167	179	63	570	577	33
90	103	92	173	184	62	606	612	32
91	104	91	177	189	61	644	652	31
93	106	90	183	194	60	688	694	30

* Laying Condition A: flat-bottom trench, without blocks, untamped backfill.
† Laying Condition B: flat-bottom trench, without blocks, tamped backfill.
‡ The $\frac{D}{t}$ for the tabulated $\frac{W_e}{D}$ nearest to the calculated $\frac{W_e}{D}$ is selected. When the calculated $\frac{W_e}{D}$ is halfway between two tabulated values, the larger $\frac{W_e}{D}$ should be used.

SOURCE: American Standard for Thickness Design of Ductile Iron Pipe (A21.50-1965).

THICKNESS DESIGN OF DUCTILE-IRON PIPE

TABLE 6

Standard Thickness Classes of Ductile-Iron Pipe

Size in.	Thickness Class 1	2	3	4	5	6
	Thickness—*in.*					
3		0.28	0.31	0.34	0.37	0.40
4		0.29	0.32	0.35	0.38	0.41
6		0.31	0.34	0.37	0.40	0.43
8		0.33	0.36	0.39	0.42	0.45
10		0.35	0.38	0.41	0.44	0.47
12		0.37	0.40	0.43	0.46	0.49
14	0.36	0.39	0.42	0.45	0.48	0.51
16	0.37	0.40	0.43	0.46	0.49	0.52
18	0.38	0.41	0.44	0.47	0.50	0.53
20	0.39	0.42	0.45	0.48	0.51	0.54
24	0.41	0.44	0.47	0.50	0.53	0.56
30	0.43	0.47	0.51	0.55	0.59	0.63
36	0.48	0.53	0.58	0.63	0.68	0.73
42	0.53	0.59	0.65	0.71	0.77	0.83
48	0.58	0.65	0.72	0.79	0.86	0.93

SOURCE: American Standard for Thickness
Design of Ductile Iron Pipe
)A21.50-1965).

LECTURE ON WATERMAIN APPURTENANCES

PART 3 APPURTENANCES TO WATERMAINS

1. Joints

There are many ways of joining two pieces of pipe together. I will not attempt to explain in detail all the different types of joints available but only those most commonly used.

a) Mechanical Joint

The mechanical joint is an adaption of the stuffing-box principle. It consist of a socket, or special bell, provided with a flange cast integral with it; a cast iron gland or follower ring, a rubber gasket, and necessary bolts and nuts. Perusal of the cross-sectional drawing shown on page 20 will indicate that tightening of the bolts results in compression of the rubber gasket and thereby a sealing of the pipes.

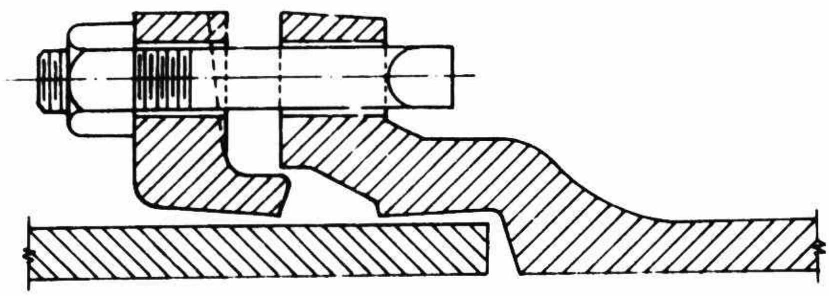
Mechanical joint cast iron or ductile iron pipe offers many advantages in addition to providing a bottle-tight joint for all working pressures. It permits considerable deflection as well as longitudinal expansion and contraction in line without causing leakage. For example, an 8-inch diameter watermain can be deflected up to 5 degrees at the joint to assist in changing the alignment of the main. In addition, the joint is relatively easy to assemble and requires only a ratchet wrench for assembly.

The joint is primarily used for connecting fittings such as tees, wyes, crosses, valves and bends to cast iron and ductile iron watermains. There is also a mechanical joint available for asbestos-cement watermains.

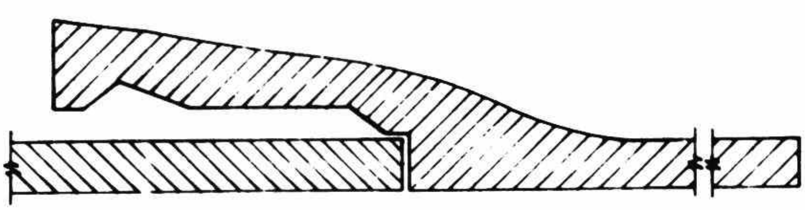
b) Push-on Joints

A push-on joint is primarily a bell and spigot rubber gasket joint. Again the rubber gasket is compressed to provide a seal between the two pipes. Perusal of the sketch on page 20 indicates that a longitudinal push on the pipe causes the necessary compression of the ring to join the two pipes together.

Since only one component is required for cast iron and ductile iron pipe the joint provides easy and rapid assembly. Again, as in the mechanical joint, the pipe may be deflected up to 5 degrees to permit changes in watermain alignment.



STANDARD MECHANICAL - JOINT
for
CAST and DUCTILE IRON PIPE



STANDARD BELL and SPIGOT JOINT
for
CAST and DUCTILE IRON PIPE

b) Push-on Joints-Continued:

A similar type of joint is available for asbestos-cement pipe. This joint consist of a separate outer ring equipped with two gaskets to provide compression against both pieces of pipe being joined.

Similar joints are available for reinforced concrete and steel pipe.

c) Dresser Couplings

Dresser Couplings are a form of mechanical joints. They consist of a middle ring with a centre stop preventing adjacent pipe ends from butting end to end; rubber gaskets, and follower glands tied together with bolts.

d) Victaulic Couplings

Victaulic Couplings are a form of mechanical joint consisting of a bolted, segmental, clamp type mechanical coupling whose housing encloses a rubber gasket. The housing locks the pipe end together to prevent end movement, yet allow some degree of flexibility and alignment. Ends of pipe must be especially prepared to accommodate Victaulic couplings. This is done by grooving, banding, rolling, or welding adapters to pipe ends.

e) Other methods of Jointings

Field welding of water pipe joints is used quite extensively on steel pipe 24 inches in size and over. Welded joints provide permanent tightness and strength. Several kinds of field welded joints are satisfactory including single welded butt joints for lap welding, single and double welded butt joints.

In addition, no thesis on jointing of watermain pipe is complete unless mention is made of lead jointing. In by gone years this was the predominent method of jointing cast iron pipe.

2. Tees and Crosses

Tees and Crosses,as their name implies, provides for three-way and four-way branches. Cast iron fittings, with various joints, are available for cast iron, ductile iron, and asbestos-cement pipe.

2. Tees and Crosses - Continued

American Standard A21.10 - 1964, which also bears AWWA number C110-64, is available for cast iron fittings in the range of 2-inch to 48-inches. This Standard provides for mechanical joints, push-on joints and flanged joints. In addition, it is possible to obtain these fittings with one end as a plain end to provide for the bell end of an adjoining pipe.

All tees, and in some cases crosses, require blocking to prevent longitudinal movement. It is proposed to discuss the blocking of fittings subsequently in these notes.

Tees and Crosses are normally manufactured at the plant for steel and reinforced concrete pipeline.

3. Bends

Cast iron bends are available for cast iron, ductile iron and asbestos-cement pipe for 90, 45, 22½, and 11¼ degrees. In general the same joints are available for these fittings as previously described for tees and crosses.

Bends may be manufactured for steel and reinforced concrete pipe to any angle required.

It is also necessary to provide blocking for bends to resist the thrust developed at the change of direction.

4. Reducers

It often becomes necessary to change the size of a pipeline at a particular point. It is possible to purchase cast iron reducers in a wide range of sizes covering most possibilities with respect to cast iron, ductile iron and asbestos-cement watermains. The manufacturers of steel and reinforced concrete pipe will provide shop made reducers as required.

5. End Plugs

All dead-end watermains should be capped with an end plug. Cast iron end plugs are available with various joints required to accommodate cast iron, ductile iron and asbestos-cement pipe.

It is important that end plugs be properly blocked to eliminate blow-off of the last section, or more, of pipe. The blocking should be installed in such a way as to allow for easy removal of the plug at the time it is necessary to extend the main. It is also desirable to provide a blow-off to drain the section of watermain prior to removal of the plug.

6. Valves

Valving in the distribution system is exceedingly important if reliable service is to be rendered. Valves should be spaced to minimize the area that would be deprived of water service in the event of a planned or emergency shut down. It is common practice to install three valves at a four-way intersection and two valves at a three-way intersection. It is difficult to establish any rule-of-thumb for the spacing of valves because it is dependent upon the density of land use. In both industrial and multiple family residential areas valves should be spaced closer together to minimize the impact of main shut down. In present-day residential subdivisions the tendency is towards curvilinear street patterns and a minimum number of intersections and it thus is necessary to consider the addition or deletion of valves depending upon the watermain layout.

Ordinary gate valves are most commonly used in a distribution system. These valves are lowest in cost and are usually satisfactory for ordinary operating conditions where the differential pressure between up stream and down stream sides is not great when the valve is closed. Under high differentials, these valves are frequently impossible to operate.

Although gate valves are usually of double-disc construction and are effective in stopping flow in either direction, single seat gate valves could be substituted when the pressure is in one direction only.

In specifying valves the designer must specify the direction of opening and this must be consistent with the bulk of valves presently within the system. In addition, the designer must specify the size of operating nut to be consistent with tools carried by the maintenance operation.

There are other types of valves available for use particularly with respect to main-line watermain. One of these is the Butterfly valve which should be investigated when consideration is being given to valves in excess of 16 inches. This valve is gaining in popularity due to improvementsin seat construction that have made tight closures possible in the right conditions. Consideration can also be given with respect to sizing a main-line valve of reducing the size of valve for economic purposes. The installation of a valve less than the diameter of the pipe increases the head loss through the valve and the hydraulics of this situation should be thoroughly investigated.

7. Valve Boxes and Stems

In order to permit operation of valves in a distribution system it is necessary to provide access to the operating nut. This is normally provided by the use of a valve box and stem. The stem is a hollow cylinder that fits over the operating nut. The valve box is attached to the top of the stem and provides access from the ground surface. The stem and box assembly should be adjustable to permit levelling with respect to road works.

8. Valve Chambers

Valve Chambers, or underground vaults, are usually provided for large valves. There appears to be a wide variation in the use of valve chambers and it is difficult to determine a consensus of present practice. In general it is difficult to justify valve chambers for smaller distribution valves as it rarely is necessary to disturb the valve after installation.

When it becomes necessary to design a valve chamber consideration should be given to the use of precast products. There are pre-cast bases, barrels, and top slabs available from various manufacturers. In many instances it is necessary to design and construct large chambers of reinforced concrete. The design of a valve chamber should give consideration to the purpose for which it is intended and provides sufficient space inside for that use.

9. Hydrants

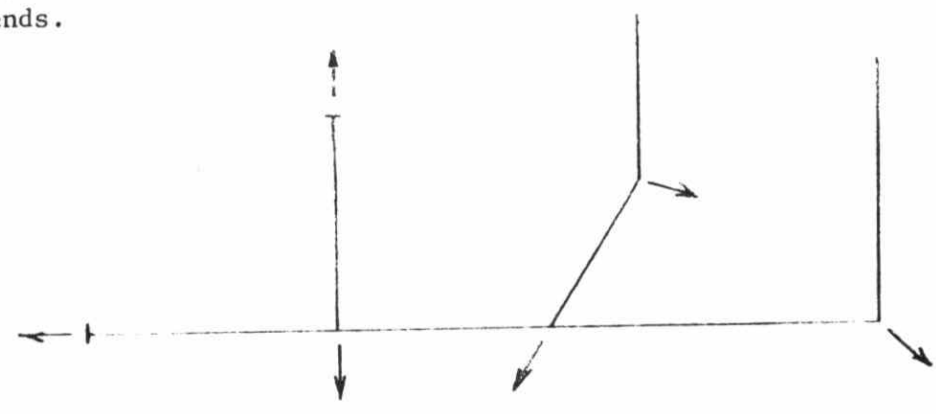
Fire protection manuals suggest that hydrants be spaced so that no dwelling is more than 300-400 feet from a hydrant. In areas of intensified land use such as industrial, commercial or multiple family residential additional thought must be given to the spacing of hydrants particularly with respect to the provision of hydrants on private property. It would be advisable to check with with your local Fire Chief with respect to the physical location of hydrants. In our instance the Fire Chief requests that hydrants be generally placed between the fire station and the area to be serviced by the hydrant. For example, on a short cul-de-sac, it is recommended that the hydrant be placed at the intersection on the station side. The logic for this recommendation is that it would require less vehicle movement than if the hydrant was placed at the end of the cul-de-sac where designers sometimes like to place it for blow off purposes.

9. Hydrants - Continued

All hydrants should be equipped with a shut off valve between the hydrant and the main. This facilitates the maintenance and replacement of hydrants without disturbance to the flow in the main. All hydrants are equipped with drainage ports and the backfill around the port should be sufficiently porous to permit drainage of water left within the hydrant after use. If the hydrant does not drain there is a possibility of the hydrant freezing in the winter and damage being done to the working mechanism. All hydrants should be properly blocked to resist longitudinal movements caused by thrust.

10. Blocking of Fittings

All pipes having joints in which longitudinal movement is not positively restricted, require external anchorages or blocking, to resist the thrust developed at changes of direction and blank ends.



The magnitude of these thrust which act in the direction shown by the arrows in the above diagram are as follows:-

for blank ends, thrust = A x P

for bends , thrust = A x P x 2 Sine ½d;

where A = area of pipe in square inches,

P = working pressure (including surge allowance if applicable) in p.s.i.,

d = angle of deviation of bend.

10. Blocking of Fittings - Continued

At average velocities the dynamic thrust due to changes in direction of the flowing water are insignificant; however, at high velocity allowances should be made for this additional thrust.

Dynamic thrust acts in the same direction as the pressure thrust and can be determined from the following equation:

$$\frac{2 \times W \times A \times V^2 \text{ sine } \tfrac{1}{2}d}{144g}$$

where W = weight of fluid in lbs/cu.ft (62.4)

G = acceleration due to gravity (32.2 feet per second per second)

V = the velocity of flow in feet per second.

The anchorages should be conservatively designed, taking into full account the maximum pressure the main is to carry on test or in service, and the horizontal holding power of the surrounding earth.

Flexible joints of the compressed gasket type offer no appreciable resistance to blow-out and anchorages should be fitted at all bends and closures if the working or test pressure exceeds a few pounds per square inch. There are many different methods available that can be used for providing the necessary blocking. Included at the end of these notes are standard drawings of various cities that are being considered by a sub-committee of the City Engineers Association and the Ontario Water Resources Commission for the basis of standard drawings to be circulated to all municipalities for their consideration as to use.

It should be noted that all blocking should be brought to undisturbed ground to get the maximum bearing pressure from the soil.

The following problems are submitted to indicate how the equations might be used.

Problem 1

The problem is to design a thrust block at the end of a 12-inch diameter watermain which is expected to operate at 80 psi, tested at 140 psi and which will not likely be subject to any significant surge. It has been determined that a safe bearing load for the undisturbed soil is 2,000 lbs per square foot.

From the equation for blank ends it can be determined that

$$\text{Thrust} = A \times P$$

$$= \frac{\pi D^2}{4} \times 140$$

$$= 15{,}800 \text{ lbs.}$$

140 psi was used as that is the highest pressure that the line will be forced to hold. The nominal diameter of the pipe is used as any greater accuracy is not needed. No allowance is made for dynamic thrust as the velocity is not usually severe.

The area of the bearing portion of the thrust block is determined as follows:

safe bearing pressure	= 2,000 lb/sq.ft.
Thrust	=15,500 lbs
Therefore area required	= 7.9 sq.ft

or approximately 2ft. 8 inches sqaure.

Problem 2

The problem is to determine the bearing area for a 90 degree bend in a 12-inch diameter watermain having the same characteristics with respect to pressure and soil as in Problem 1:

$$\text{Thrust} = A \times P \times 2 \text{ sine } \tfrac{1}{2}d$$

$$= \frac{\pi D^2}{4} \times 140 \times 2 \text{ sine } 45 \text{ degrees.}$$

$$= 22{,}400 \text{ lbs}$$

Safe bearing capacity = 2,000 lbs/sq.ft.

$$\text{Therefore area} = \frac{22{,}400}{2{,}000}$$

$$= 11.2 \text{ sq. ft.}$$

A few calculations will indicate that the bearing area required for large diameter watermains tends to become excessive. When this occurs special consideration must be given to the design of the blocking or to the tieing of joints on the pipe so that the thrust are take up by the pipe itself. There is also the danger that excavation for other services, in the street, could result in the blocking being disturbed and consideration should be given in the design of the blocking of this possibility.

11. Service Connections

Service connections can be provided by the use of copper, cast iron, ductile iron, asbestos-cement, plastic or of the other available watermain materials. The most common material used is copper, soft type K,for house connections. Various municipalities have different standards on the minimum size required for a single family residential and it is normally recommended that this size be three-quarters of an inch.

Water services should be equipped with a corporation stop and a curb stop. The corporation stop is attached to the watermain and is threaded into the wall of the main. As indicated previously consideration should be given to pipe wall thickness and tap size to ensure serviceable threaded connections. The corporation stop has a shut off so that the stop may be set in the line under pressure and subsequently open for service.

The curb stop is normally situated at the property line and provides a shut off in the event that maintenance work is required on the private service. The curb stop consists of a valve, valve stem and box similar to that on a main line valve.

All copper water services should be installed with a goose neck at the main to reduce the possibilities of ground settlement shearing the copper pipe from the corporation stop.

12. Blow off connections

Blow off connections are normally provided on large main-line watermains to provide outlets for draining the pipeline. They are usually located at dips and immediately above line valves situated on a slope. It is not customary to provide a drain for every small dip but a sufficient number of blow offs should be provided to lower the water so that a person could get through a large pipe. Short dips, such occur in practically all pipelines in City streets when a line must pass under a large drain or other structure, can often be unwatered when necessary by pumping out directly.

12. Blow off Connections - Continued

The exact location of blow off outlets is frequently influenced by opportunities to get rid of the water. Where a main crosses a stream or drainage structure, there will usually be a low point in the line, but if the main goes under the stream or drain it obviously cannot be completely drained into the channel. In such a situation, it is better to locate a blow off connection at the lowest point that would drain by gravity and provide easy means of pumping out the part below the drain flow line.

Most frequently the blow off will be below ground. As the operating nut of the valve must be accessible from the surface, the valve cannot be under the main but may be set with the stem vertical and just beyond the side of the main. The outlet from the main is most commonly constructed by bringing in a horizontal pipe tangentially to the bottom of the main. When no drain is available to receive water by gravity from the main, the water must be pumped to the surface for disposal. Blow offs and drains should be protected against freezing.

13. Air Valves

Consideration must be given to the installation of air valves on transmission watermains. Cast iron and other rigid pipes require air valves at all high points for the purpose of automatically removing air that is displaced during the filling of the line, and air that is released from the flowing water if the pressure fluctuates appreciably and if the summit lies close to the hydraulic gradient. If the pressure at the summit is high a manually operated cock or gate may be substituted because little, if any, air will accumulate and air removal is confined to filling operations.

Steel and other flexible conduits need air valves for thepurpose of automatically admitting air to the line and preventing its collapse under vacuum. A vacuum may be created when the pipe is being drained on purpose or when water escapes accidentally from the line as a result of a break at or near a low point. Air in that valve should be placed on both sides of gate valves at summits in the line, on the downstream side of other gates, and at changes in grade to steeper slopes in sections of the line that are not otherwise protected by air valves.

The required size of valves is obviously related to the size of the conduit. One recommendation is that for release of air the ratio be on the basis of 1-12 and for the admission as well as release of air on the basis of 1-8.

14. Railway Under Crossings

Pipelines crossing under railways need protection from earth movements and vibration caused by the passage of trains, while the railroads need special protection from the result from leaks or breaks in a water line under or close to the track. The most practical method of installing a pipeline under a railroad or busy highway is by threading the pipe through a culvert that has previously been jacked through without open cut.

Where ground is springy and where loads are heavy , a pipeline laid loosely in a culvert will receive very little disturbance. Pushing a pipe through a culvert introduces many hazards to the protective coating on the outside of the pipe and skids or protective lagging should be attached to be moved with the pipe. If possible, field joints should not be present in the pipe in a culvert unless they are of a type that is not expected to need future attention. If a culvert is intended to divert any leakage of water from a right-of-way, provision must be made at the ends for escape and detection of such leakage.

15. Pipe bedding

Although there is great variation in specifications adopted by municipalities and suggested by manufactureres of various types of pipe there are some consistencies. The trench bottom should be properly prepared to accept the conduit. This requires that it be accurately graded so that the pipe may be evenly supported along its length. Special care should be taken at the joints to ensure that the pipe will not be resting on the joints alone. The bottom of the trench should be cleaned and free from stones and hard lumps and if this is not economically possible consideration should be given to over excavating and backfilling with granular material. Similarly where bedrock is encountered the rock should be over excavated and the area backfilled with a granular cushion.

The side fill material should be tamped and in the case of steel watermains should be compacted to provide for the design of the pipe.

16. Freezing of watermains

In this province watermains are usually installed with 5-7 feet of cover depending upon the depth of frost penetration. The service connections, particularly those for a single family home, are most susceptible to freezing and are usually thawed electrically. It should be noted that it is not possible to thaw a plastic service electrically.

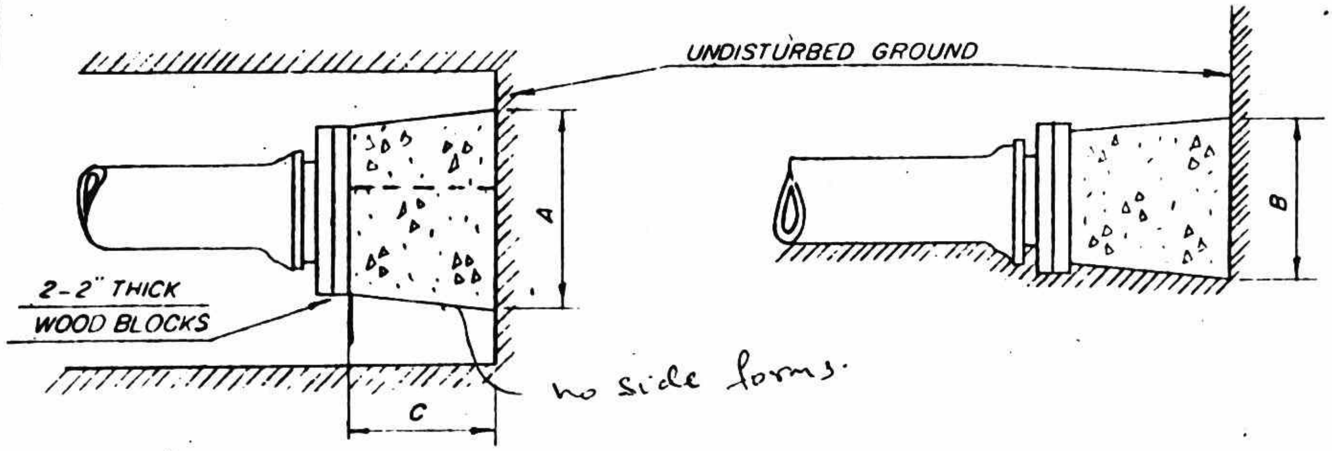
17. Layout Plans

It is not generally necessary to have exact dimensions with respect to working with cast iron, ductile iron and asbestos-cement pipe as it is possible to work in the various fittings by cutting the pipe in the field. Watermain pipe should be laid to a designed grade so that proper record can be obtained of their vertical position.

The layout design for steel and reinforced concrete pipe is more involved due to the fact that fittings and closure pieces are pre-manufactured to suit. The Steel Handbook sets out the following:

> In general, a plan of profile, together with certain other details are necessary for any water pipeline. These should show:
>
> 1. Horizontal and vertical distances, either directly or by survey station and elevation.
> 2. Location of angle or bend, both horizontal and vertical (point of interesection preferred).
> 3. Degree of bends, degree or radius of curves, tangent distances for curves, or external distances if clearance is required.
> 4. Points of intersection with pipe centre line for tees, wyes, crosses, or other branches, together with direction - right or left hand, up or down - or angle of flow, viewed from inlet end.
> 5. Location and covering length of all valves, pumps, or other inserted fittings not supplied by the pipe manufacturer.
> 6. Location of adjacent or interfering installations or structures.
> 7. Tie-ins with property lines, curb lines, road or street centre lines, and other pertinent features necessary to design righ-of-way and locate pipe centre-line clearly.
> 8. Details of descriptions of all specials, together with data required to supplement AWWA Standards.
> 9. Details, dimensions, and class designation or other descriptions of all flanges and mechanical field joints.
> 10. Any special requirements affecting the manufacturer of the pipe or any installation procedure.

SCARBOROUGH PUBLIC UTILITIES COMMISSION



PLAN

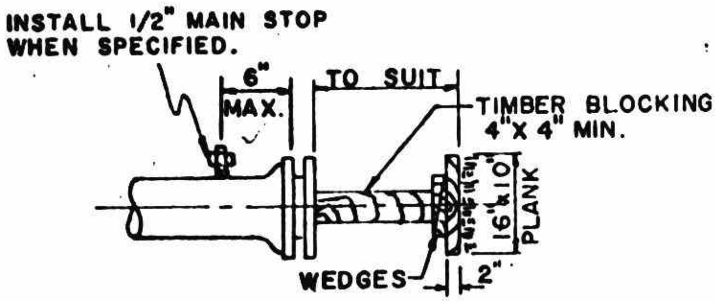
ELEVATION

- NOTES -

1) MINIMUM STRENGTH CONCRETE 2500 P.S.I

2) WHERE WATER PRESSURE EXCEEDS 100 P.S.I. INCREASE MINIMUM DIMENSIONS BY 10% EVERY 10 P.S.I.

~~from Temporary plugs~~



PLUGS 4"- 12" WATERMAIN
TEMPORARY

1-1

MINIMUM DIMENSIONS AT MAXIMUM 100 P.S.I
WATER PRESSURE

SIZE OF PIPE	'A'	'B'	'C'
6"	18"	16"	18"
8"	24"	16"	24"
10"	30"	20"	30"
12"	36"	24"	36"
16"	48"	36"	48"

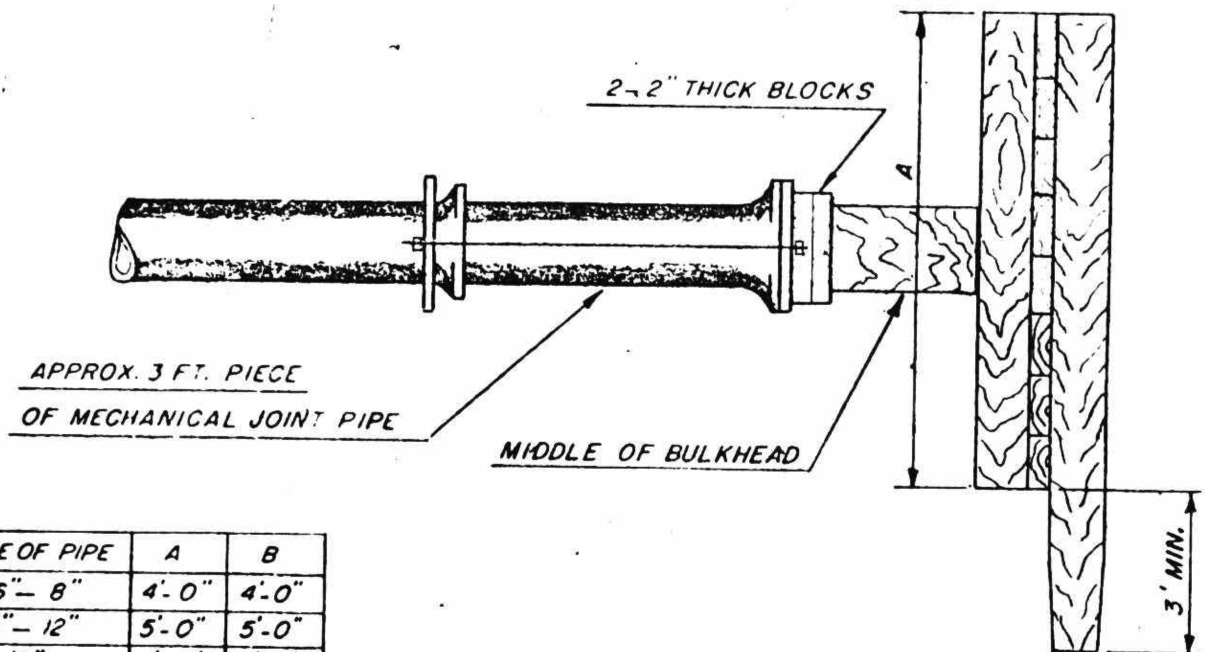
TITLE

	NAME	DATE
DRAWN BY	E.W.S.	8-3-66
CHECKED	R.H.T.	27-4-66
APPROVED	KUa.	28-4-66

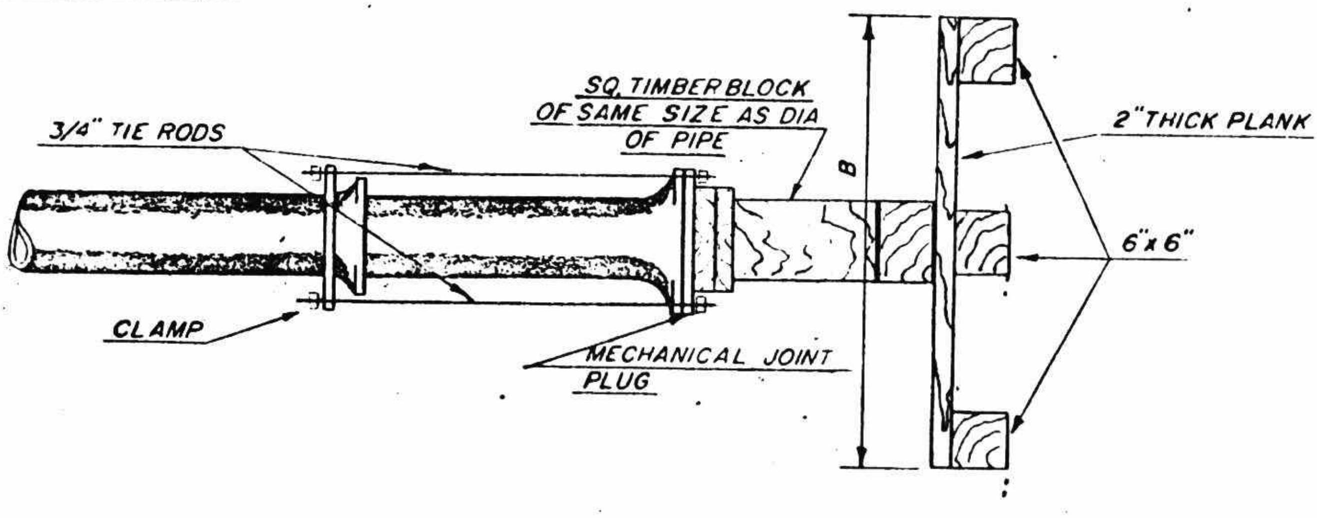
~~CONCRETE~~ THRUST BLOCKS FOR PLUGGED TEES, CROSSES & DEAD END MAINS

DWG. NO. WS-5

SCARBOROUGH PUBLIC UTILITIES COMMISSION



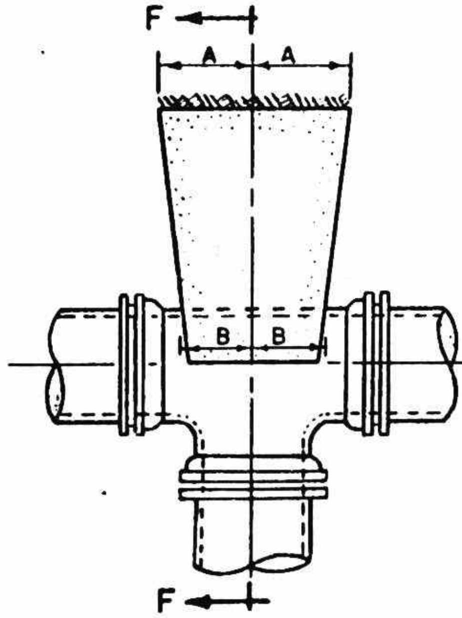
SIZE OF PIPE	A	B
6" – 8"	4'-0"	4'-0"
10" – 12"	5'-0"	5'-0"
16"	6'-0"	6'-0"

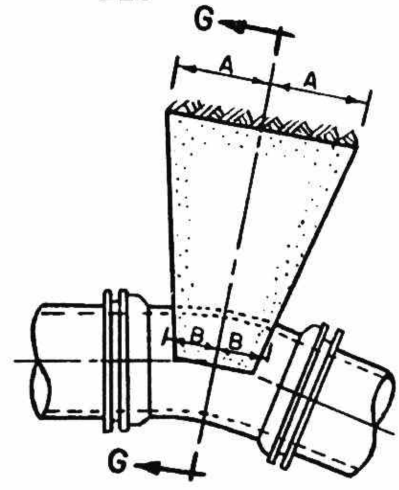


1 - 2

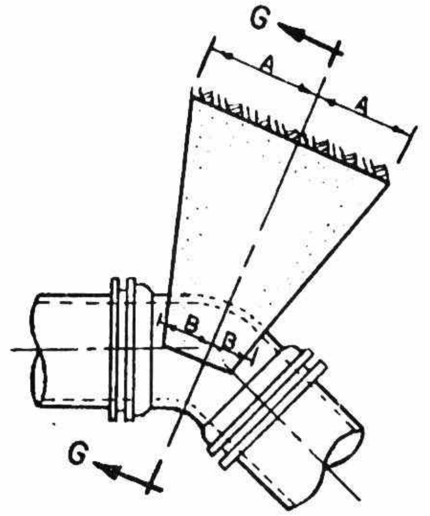
	NAME	DATE	TITLE
			~~BULKHEAD DETAILS~~ FOR BLOCKING
DRAWN BY	E.W S	5-3-66	DEAD-END WATERMAIN IN FILL
CHECKED	AHT	27-4-66	GROUND
APPROVED	KHa	28-4-66	DWG. NO. WS-6

Toronto - Identical.

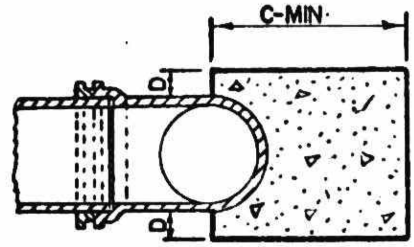




CONC· THRUST BLOCK
FOR 1/16 & 1/32 BEND

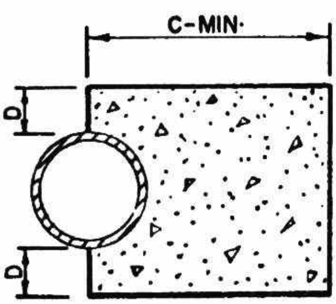


CONC· THRUST BLOCK
FOR 1/8 BEND

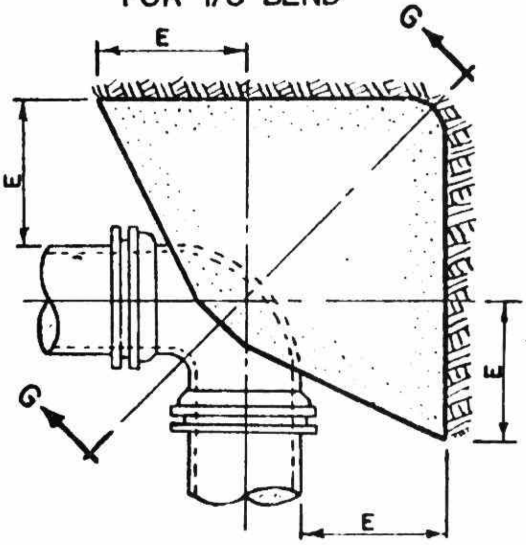


SECTION F-F

CONC· THRUST BLOCK
FOR TEE BRANCH



SECTION G-G



CONC· THRUST BLOCK
FOR 1/4 BEND

16 = ??

NOTES:

1· ALL CONCRETE TO BE 2500 P·S·I·

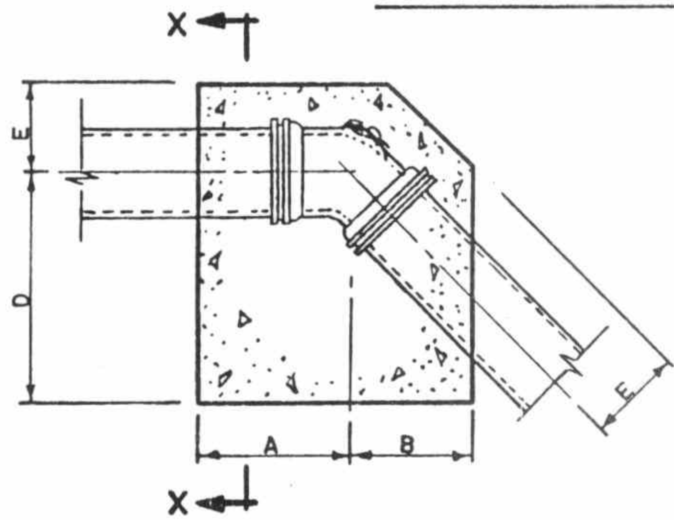
2· ALL CONCRETE TO BE AGAINST UNDISTURBED TRENCH WALL

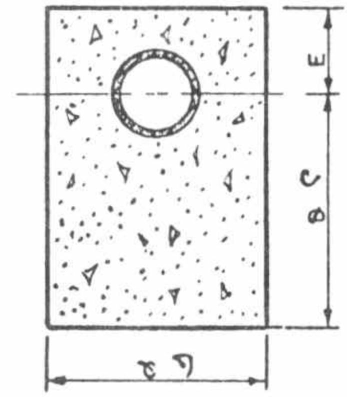
(1-3)

	TEE BRANCH					1/4 BEND					1/8 BEND					1/16 & 1/32 BEND				
PIPE DIA·	6"	8"	10"	12"	16"	6"	8"	10"	12"	16"	6"	8"	10"	12"	16"	6"	8"	10"	12"	16"
A	6"	8"	10"	1'-0"	1'-6"						9"	9"	1-0"	1'-3"	1'-8"	6"	9"	1'-0"	1'-3"	1'-8
B	3"	5"	7"	9"	1'-0"						3"	4"	5"	6"	9"	3"	4"	5"	6"	9"
C-MIN·	1'-6"	1'-10"	2'-0"	2'-6"	2'-9"	1'-6"	1'-10"	2'-0"	2'-6"	3'-3"	1'-6"	1'-10"	2'-0"	2'-6"	2'-9"	1'-6"	1'-10"	2'-0"	2'-6	2'-9"
D	3"	3"	3"	3"	3"	3"	4"	5"	6"	6"	3"	4"	5"	6"	6"	3"	4"	5"	6"	6"
E						1'-0"	1'-2"	1'-4"	1'-6"	2'-0"										

BOROUGH OF EAST YORK ENGINEERING DEPARTMENT	DWG. BY M·MURRAY SCALE 1/2"=1'-0"	STANDARD HORIZONTAL CONCRETE THRUST BLOCKS FOR WATER MAINS	STA· 158
WORKS COMMISSIONER:	FILE:		FEB·20/69

VERTICAL BEND DOWN



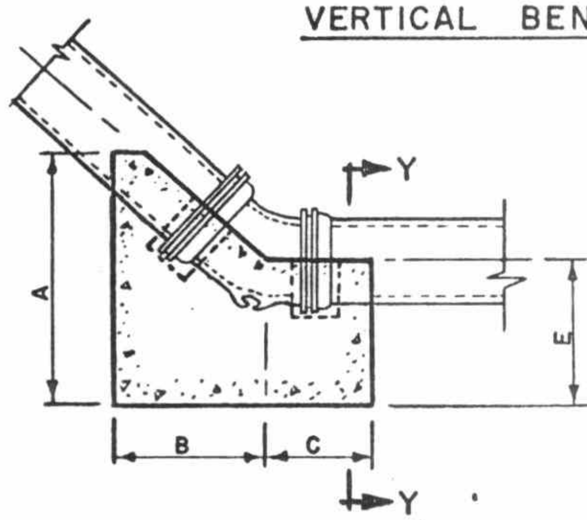


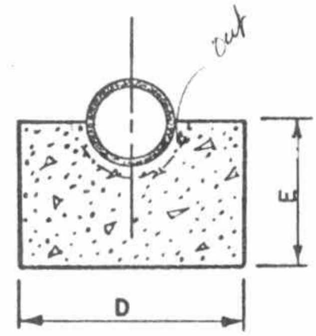
SECTION X-X

NOTE:
ALL CONCRETE TO
BE ~~2500~~ P.S.I.
3000

	1/8 BEND					1/16 & 1/32 BEND				
PIPE DIA.	6"	8"	10"	12"	16"	6"	8"	10"	12"	16"
A	1'-0"	1'-0"	1'-3"	2'-0"	2'-0"	1'-0"	1'-0"	1'-3"	1'-9"	1'-9"
B	1'-0"	1'-3"	1'-6"	1'-9"	2'-0"	1'-0"	1'-0"	1'-3"	1'-9	1'-9
C	2'-0"	2'-0"	2'-6"	3'-0"	3'-0"	2'-0"	2'-0"	2'-6"	3'-0"	3'-0"
D	1'-8"	1'-8"	2'-0"	3'-0"	3'-0"	1'-3"	1'-3"	1'-6"	2'-6"	2'-6"
E	9"	10"	1'-0"	1'-3"	1'-6"	9"	10"	1'-0"	1'-3"	1'-6"

VERTICAL BEND UP





SECTION Y-Y

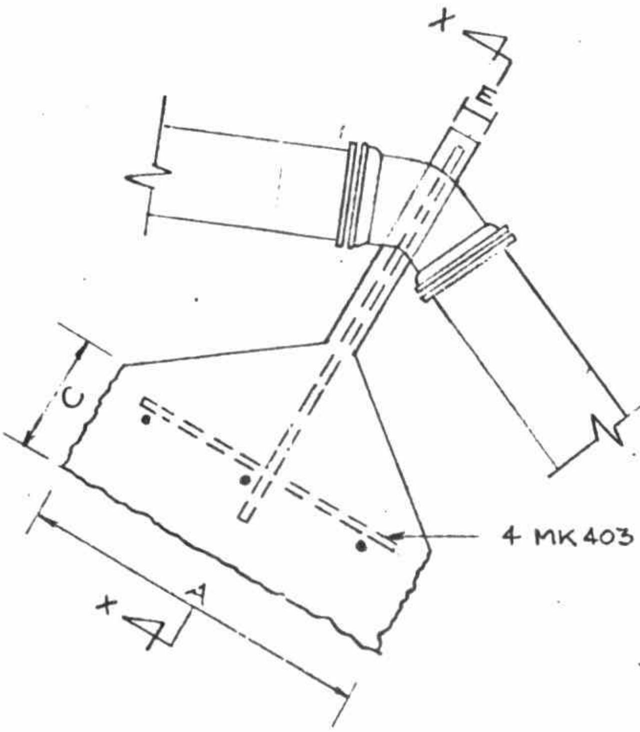
(3-4)

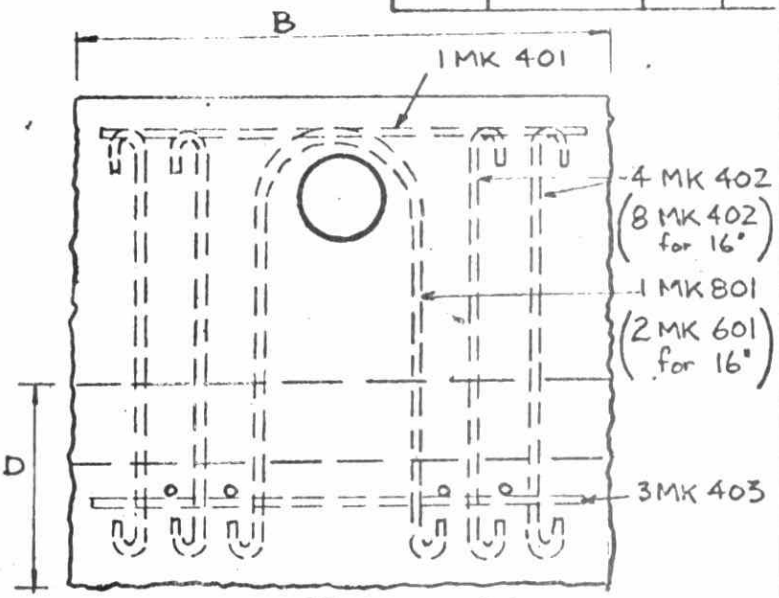
NOTES:
1. ALL CONCRETE TO BE 3 ~~2500~~ P.S.I.
2. ~~CONCRETE TO BE 2" MINIMUM CLEAR AROUND BELLS & GLANDS~~

	1/8 BEND					1/16 & 1/32 BEND				
PIPE DIA.	6"	8"	10"	12"	16"	6"	8"	10"	12"	16"
A	2'-0"	2'-0"	3'-0"	3'-6"	3'-6"	1'-6"	1'-6"	1'-9"	2'-3"	2'-3"
B	1'-3"	1'-3"	1'-6"	2'-6"	2'-6"	1'-3"	1'-3"	1'-6"	1'-9"	1'-9"
C	1'-0"	1'-0"	1'-3"	1'-6"	1'-6"	1'-0"	1'-0"	1'-3"	1'-6	1'-6"
D	2'-0"	2'-0"	2'-6"	3'-0"	3'-0"	2'-0	2'-0"	2'-6"	3'-0"	3'-0"
E	1'-3"	1'-3	1'-6"	2'-0"	2'-0"	1'-3"	1'-3"	1'-6"	2'-0"	2'-0"

BOROUGH OF EAST YORK ENGINEERING DEPARTMENT	DWG. BY M.MURRAY	STANDARD VERTICAL CONCRETE ANCHOR BLOCKS FOR WATER MAINS	STA. 159
WORKS COMMISSIONER:	SCALE FILE:		FEB. 24/69

C.E.A. - O.W.R.C. MODEL No.	WS-202
Date	Rev.

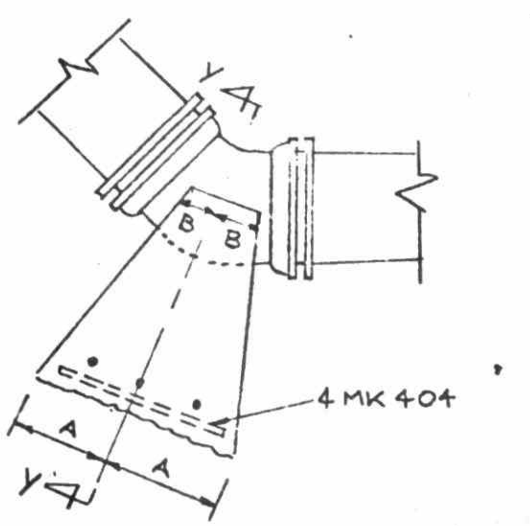


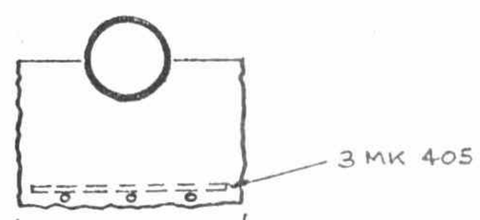


SECTION XX

	1/8 BEND					1/16 & 1/32 BEND				
PIPE DIA	6"	8"	10"	12"	16"	6"	8"	10"	12"	16"
A	2'-6"	3'-6"	5'-0"	5'-0"		2'-0"	2'-6"	2'-6"	3'-6"	4'-0"
B	3'-6"	4'-0"	5'-0"	5'-0"		2'-0"	3'-6"	4'-0"	4'-9"	5'-0"
C	1'-1"	1'-1"	1'-1"	1'-1"		1'-0"	1'-4"	1'-4"	1'-4"	1'-6"
D	1'-6"	1'-6"	2'-0"	2'-0"	2'-6"	1'-0"	1'-6"	1'-6"	1'-6"	1'-6"
E	5"	5"	6"	6"	9"	8"	8"	8"	1'-0"	1'-0"

NOTE: 1. CONCRETE 3000 psi





SECTION YY

NOTE: 1. USE REINFORCING FOR 12" & UP ONLY

	1/8 TO 1/32				
PIPE DIA.	6"	8"	10"	12"	16"
A	9"	9"	1'-0"	1'-3"	1'-8"
B	3"	4"	5"	6"	9"

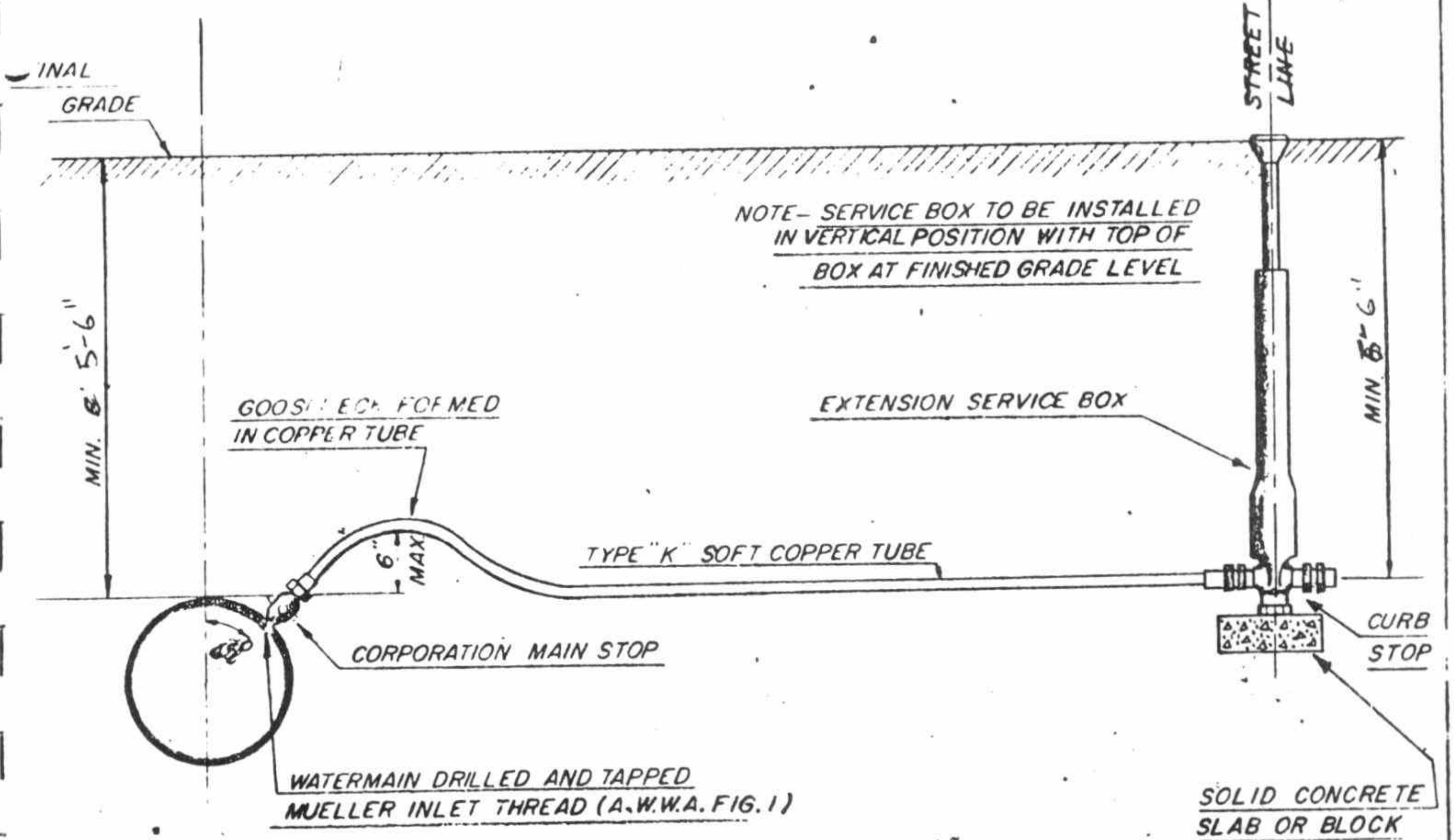
MUNICIPALITY

VERTICAL ANCHOR BLOCKS

- REINFORCED -

APPROVED

SCARBOROUGH PUBLIC UTILITIES COMMISSION

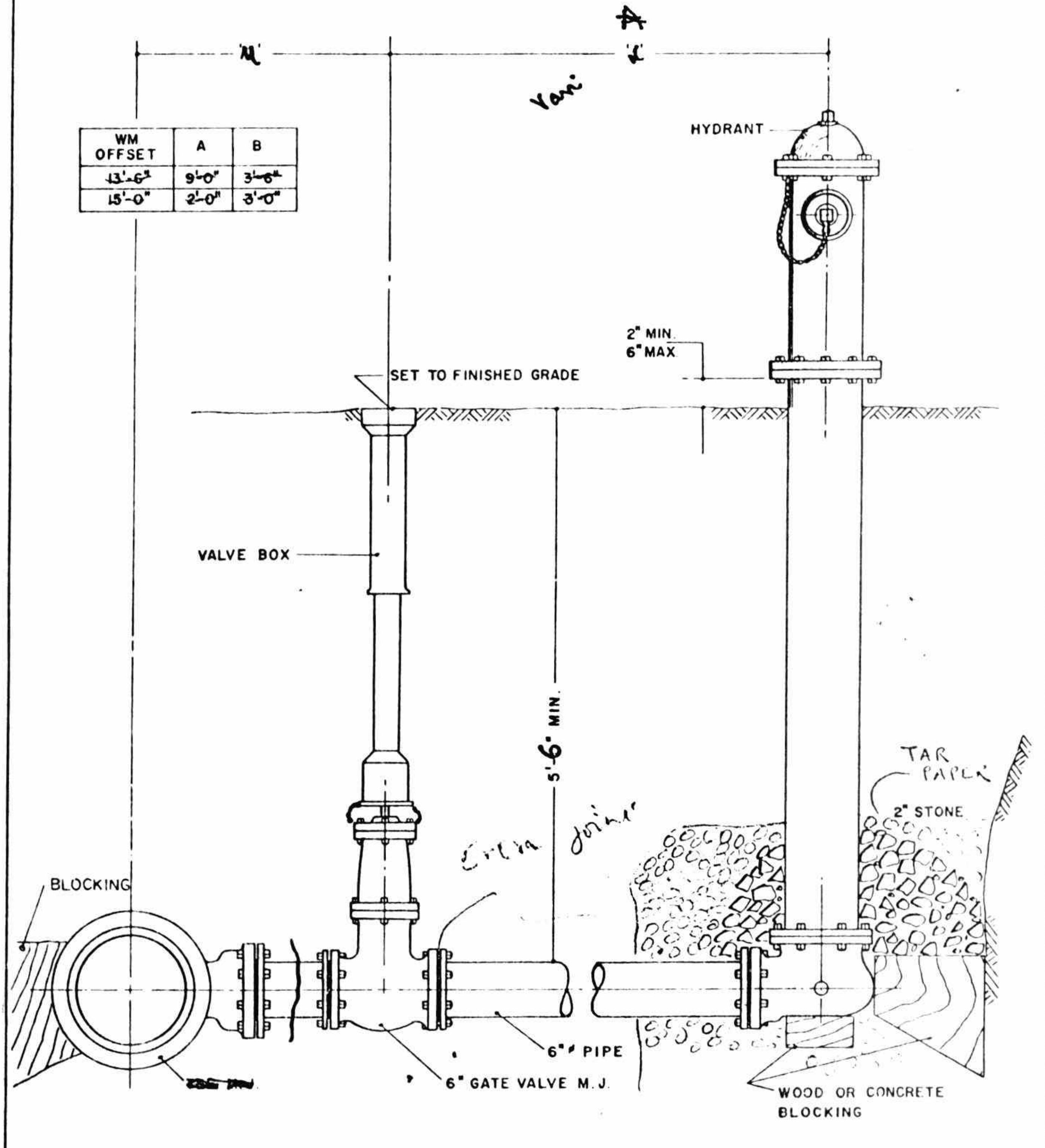


-NOTE-

1) ANY JUNCTION MADE IN SERVICE PIPE BETWEEN CORPORATION MAIN STOP AND CURB STOP TO BE MADE WITH CORPORATION COUPLING.
2) CURB STOP ONLY TO BE INSTALLED ON WATER SERVICES SIZES 3/4" TO 1" INCLUSIVE. 5/8 2"
3) FOR SERVICES LARGER THAN 1" A MUELLER "MARK II" ORISEAL VALVE, OR EQUAL IS REQUIRED IN PLACE OF A REGULAR CURB STOP.
4) SERVICES LARGER THAN 1" REQUIRE A 2" SERVICE BOX.
5. ALL WATER SERVICES TO BE INSTALLED AT RIGHT ANGLES TO THE WATERMAIN UNLESS APPROVED OTHERWISE.

4-1

REVISION, NOTE (3) REVISED 2-20-69			TITLE
REVISION 5/8" REVISED to 3/4" FEB 29-68			STANDARD WATER SERVICE CONNECTION DETAILS FOR SIZES 3/4" TO 2" INCLUSIVE
	NAME	DATE	
DRAWN BY	E. W. S	6-3-66	
CHECKED	AHT	27-4-66	
APPROVED	[illegible]	28-4-66	DWG. NO. WS-3



NOTE:

THE PUBLIC UTILITIES COMMISSION
OF THE TOWN OF MISSISSAUGA

STANDARD HYDRANT SETTING

(6.-1)

DATE	REVISIONS	
Nov./69	SUPERSEDES DWG. DATED MAY/67	SCALE N.T.S.
May/70	DIMENSION A & B	DATE OF PLAN Nov./69
		DWG. BY R. KELERIS
		STD. DWG. S-5

PREDICTING FLOWS AND PHYSCIAL FEATURES

OF

SEWER SYSTEMS

by

Mr. P. A. Harris, P. Eng.

July 20, 1972.

PREDICTING FLOWS FOR STORM AND SANITARY SEWERS

PART 1 - STORM SEWERS

A. INTRODUCTION

The main purpose for storm sewers is the rapid and effective removal of storm water from places where it is not wanted, and the direction of this water to a suitable outlet, usually a creek, river or lake.

Municipal engineers involved in the design of such sewers must be aware of both the hydraulic elements, and the hydrology affecting the design. The hydraulic elements include the determination of the sizes, depths, shapes and materials of the proposed sewer construction.

The hydrologic phase of urban design involves the determination of magnitude, distribution and timing of various run-off events. Run-off is the part of rainfall not lost by infiltration into the soil or left in ground depressions and on vegetation surfaces to evaporate. A number of the factors that affect run-off are: climate; physical characteristics of the particular locality; rainfall intensity, duration and distribution; initial soil moisture conditions; soil type; the shape, size and slope orientation of the rainfall area; and the land use characteristics of the general drainage area.

THE DESIGN FLOW

Peak flows result from excess surface run-off volumes. The conditions which may generate these excesses are intense storms, snow melt, or snow melt combined with rainfall. Maximum flows on urban areas usually result from high intensity short-duration rainfall of the thunderstorm type.

The "design flow" may be defined as the maximum flow which a specified structure can pass safely.

PROCEDURES FOR ESTIMATING RUN-OFF

Procedures used in estimating run-off magnitude and frequency have greatly improved in the last twenty to thirty years and can be generally categorized as: (1) empirical approaches, (2) statistical or probability methods, and (3) methods relating rainfall to run-off.

Of the three approaches previously mentioned, the third one, i.e. to relate rainfall to run-off is the most widely accepted method and can be applied to both the "unit-hydrograph" method and the "rational method". Since nearly every municipality in Canada uses the "rational method" the rest of this discussion will be used to explain its use.

THE "RATIONAL METHOD"

This method, first introduced in 1889, relates rainfall to run-off by the equation: -

$$Q = A\ i\ R \quad \text{where}$$

Q = peak run-off in c.f.s. (cubic feet per second)

R = run-off coefficient

i = average rainfall intensity (in./hr.) for a period equal to the time of concentration

A = drainage area in acres

Two basic assumptions which must apply when using this method are: (1) the maximum run-off rate for any design location is a function of the average rate of rainfall during the time of concentration and (2) the maximum rate of rainfall occurs during the time of concentration. The time of concentration, tc, is defined as the flow time from the most remote point in the drainage area to the point in question.

It should be pointed out here that this method relies on average rainfall intensities prevailing over the time of concentration and that these average intensities have no relation to the actual rainfall pattern during the storm.

We should also note that the "rational method" is generally applicable to urban areas of less than 5 sq. miles and for larger areas, the application of the hydrograph method is usually warranted.

Now let's take a closer look at each of the components in the $Q = C\ i\ A$ formula.

EXPECTED FLOW - "Q"

It is noted here that the assignment of the units c.f.s. (cubic feet per second) is satisfactory to this equation since 1.008 c.f.s. equals one inch of rainfall in one hour over an area of one acre.

RUN-OFF COEFFICIENT - "R"

This coefficient can be further defined as the ratio between the maximum rate of run-off from the area and the average rate of rainfall on the area during the time of concentration. This coefficient cannot be exactly determined since it includes the influence of a number of variables, such as infiltration capacity of the soil, depression storage and interception by vegetation. The closer the area comes to being impervious, the more reasonable the selection of "C" becomes, since for highly impervious area "R" approaches unity. For these reasons mentioned, the run-off coefficient "R" is the component of the rational formula which requires the greatest exercise of judgement by the design engineer.

A partial list of coefficients presently in use are:

Character of Surface	Run-off Coefficients
Asphalt	0.70 - 0.95
Drives and Walks	0.75 - 0.85
Lawns - sandy soil	0.10 - 0.20
Lawns - heavy soil	0.13 - 0.30

Often, municipalities will use an "average" coefficient, where more than one type of surface is present in a particular drainage area. Some of those in use are:

Description of Area	Run-off Coefficients
Single Family	0.30 - 0.50
Semi-detached	0.40 - 0.50
Apartments	0.50 - 0.75
Industry	0.50 - 0.90
Parks, green belt	0.10 - 0.25

These coefficients are widely used for storms of 5 year to 10 year frequencies. Less frequent, higher-intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on run-off.

RAINFALL INTENSITY - "i"

Rainfall behaviour varies with location and topography. The coefficient "i" is commonly determined from an equation such as:

$$i = \frac{x}{t + y} \quad \text{where}$$

i = rainfall intensity, in./hr.

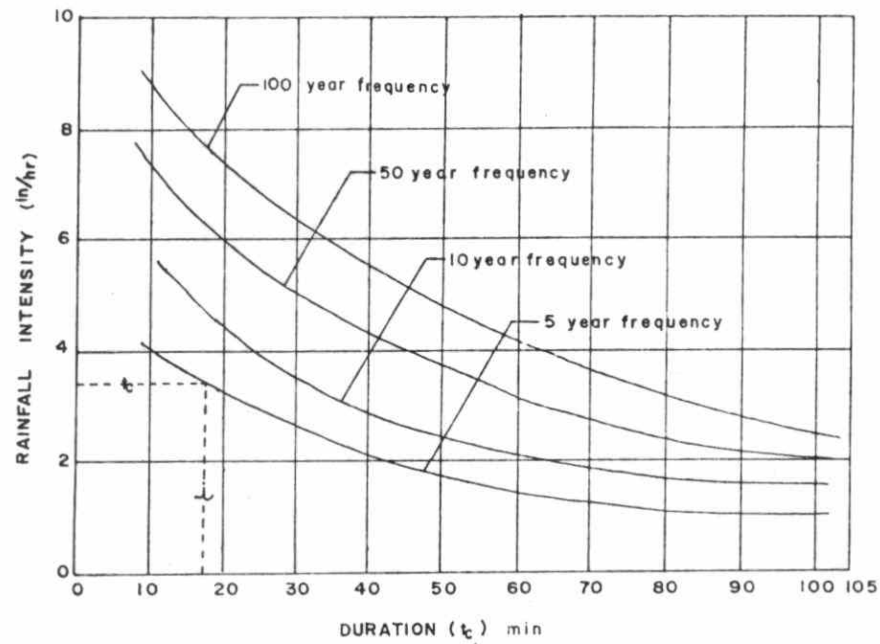
t = time of concentration

x, y = constants determined from previous records.

A well designed drainage system provides maximum protection at minimum costs. An economic balance is necessary between the cost of structures and the direct and indirect cost of public damage to property and inconvenience to the public over a long period of years. In areas where the effects of minor flooding would not be damaging, the system may handle ordinary storms but insufficient to handle infrequent and more intense storms. On the other hand, in valuable city and highly built-up areas, where even minor flooding could be disastrous and very costly, the system must be designed to handle the more severe storms.

Drainage systems are designed to handle storms having frequencies of two years to ten years, depending on the value of the property. In commercial areas and for flood-wall structures this frequency is often as high as 50 years. For purposes of design the rainfall frequency is usually established by each municipality. Design frequency, as the words are used here, is the frequency with which a given event is equalled or exceeded on the average, once in a period of years. Thus a ten year frequency event would be expected to be equalled or exceeded ten times in 100 years.

The relationship $i = \frac{x}{t + y}$ is hyperbolic. In practice, there is usually an upper limit on "i" which corresponds to a value of tc from 10-15 minutes in urban areas and as low as 5 minutes for industrial or highly built-up areas. A typical set of intensity - duration (tc) frequency curbes would appear as below: -



For urban storm sewers, the time of concentration consists of inlet time, or the time required for run-off to flow over the surface from the most remote point in the drainage area to the nearest inlet, plus the time of flow in the sewer from the uppermost point in the sewer to the point in consideration. The time of flow in the sewers is usually the average full-flow velocity expected for the particular sewer size, shape and roughness factor. (i.e. hydraulics of the sewer). As shown on the previous figure, for a 5 year storm, and a tc of 15 minutes, the intensity would be 3.5± in./hr.

AREA - "A"

The drainage area "A" is that area of land which is tributary to any particular point under consideration. The entire drainage area chosen by prior topographical surveys or watersheds, municipal or political boundaries, must be subdivided in sub-areas tributary to each inlet point.

The initial sewer pattern or layout is seldom final, as often much re-organization of the component parts is required to determine the best points of inlet, the times of concentration to these points and the effects of alternative routes to the entire drainage pattern. In the preliminary drainage area studies, the designer must be concerned with:

1) land use - present and future,

2) percentage of imperviousness of inlet areas,

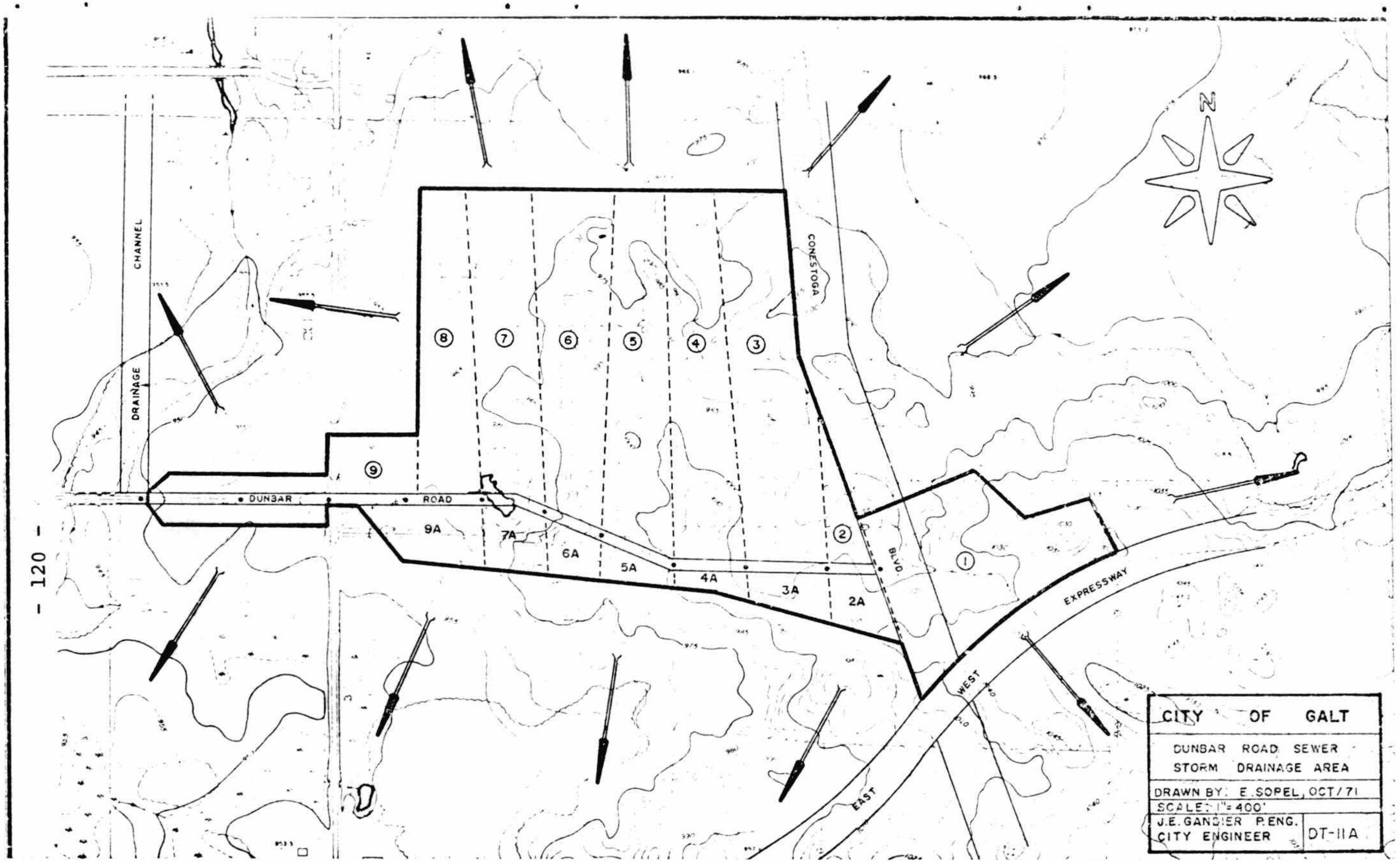
3) character of soil and cover,

4) ground slopes and shapes of the drainage area.

Attached to your papers is a small storm sewer design drainage area map, rainfall intensity curve and circular pipe monograph. It is easy to follow starting with area 1 at the east limit of this particular drainage area and working to the west limit. Areas 2 through 7 are split into two sections for acreage purposes but could have been combined as individual area. Listing the areas, run-off coefficients and calculating the 'A x R' and 'accumulated A x R' values is the first step in preparing the design sheet. The 'time of concentration' or 'inlet time' is usually set by each municipality and often varies from area to area. As shown on the design sheet, the inlet time has been set at fifteen minutes. From our rainfall

intensity curve, the corresponding 'intensity' for a time of concentration of 15 minutes is 3.77 in./hr. Multiplying the 'accumulated A x R' by this intensity gives us an expected flow of 30.91 c.f.s. It is then a simple matter of selecting a pipe size and grade to handle this flow. This selection can be made from the circular pipe nomograph or from prepared charts. Through each pipe section we calculate the time of flow and add it to the time of concentration for the next section. From the intensity curve you can see that as the time increases the intensity decreases.

As was mentioned earlier, in the rational method, average intensities have no time sequence relation to the actual rainfall pattern during the storm. In spite of the limitations of this method, experience has found it to yield satisfactory results if the designer uses his judgement in evaluating the component factors. Its use can be compared to the almost primitive, but still popular method of setting boning rods or batter boards for installing sewers to proper line and grade.


CHANNEL
DRAINAGE
DUNBAR
ROAD
CONESTOGA
BLVD.
EXPRESSWAY
WEST
EAST
N
(1)
(2)
(3)
(4)
(5)
(6)
(7)
(8)
(9)
2A
3A
4A
5A
6A
7A
9A
CITY OF GALT
DUNBAR ROAD SEWER
STORM DRAINAGE AREA
DRAWN BY: E. SOPEL, OCT/71
SCALE: 1" = 400'
J.E. GANDIER P. ENG.
CITY ENGINEER
DT-11A

CITY OF GALT

STORM SEWER DESIGN CHART

STD. 8-1(A)

SUBDIVISION Dunbar Road

DRAINAGE AREA PLAN Nº DT-11A

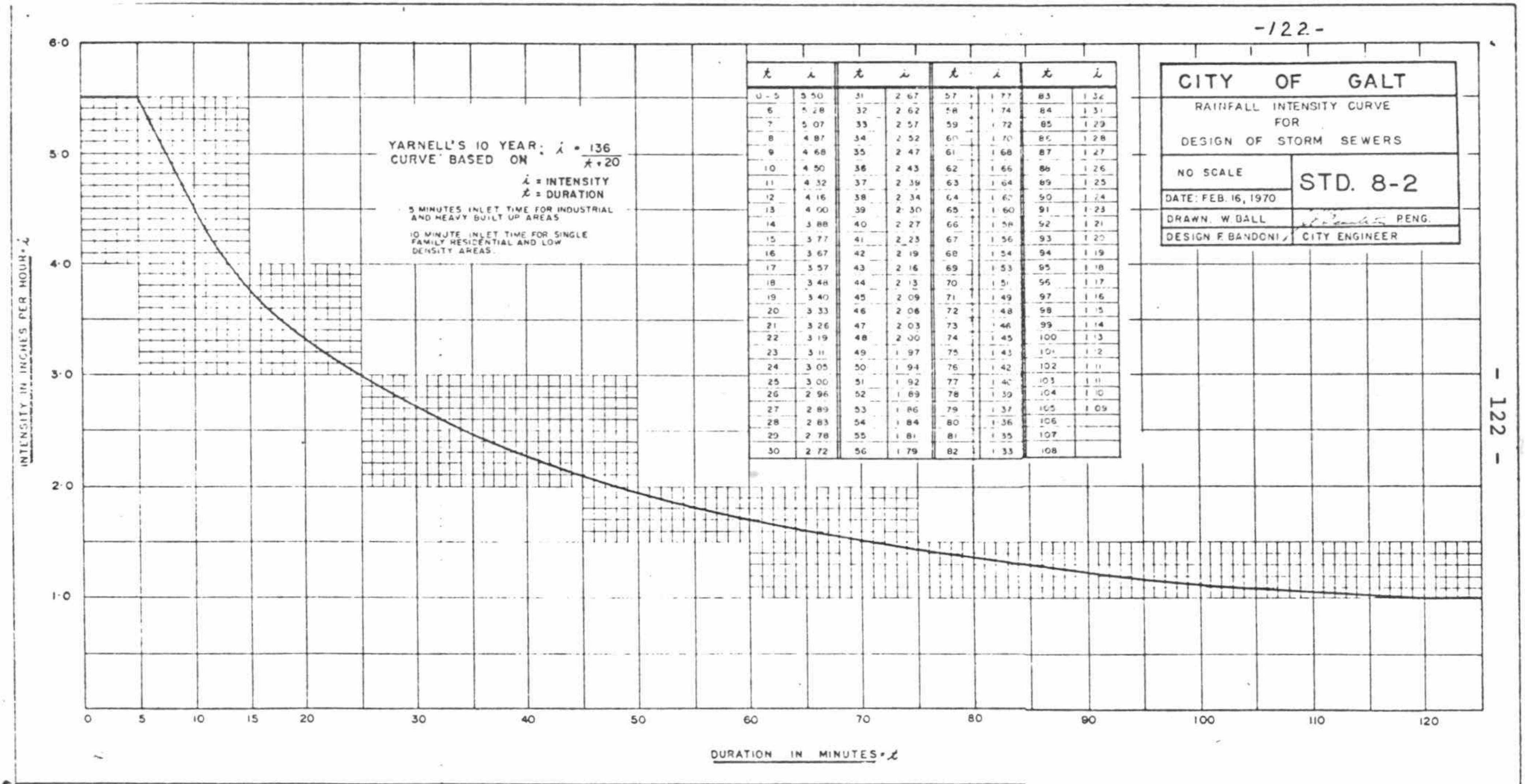
SHEET Nº 1 OF 1

PROJECT Nº 71-14 n= 0.013

DESIGNED BY P.H. DATE Oct.1/71

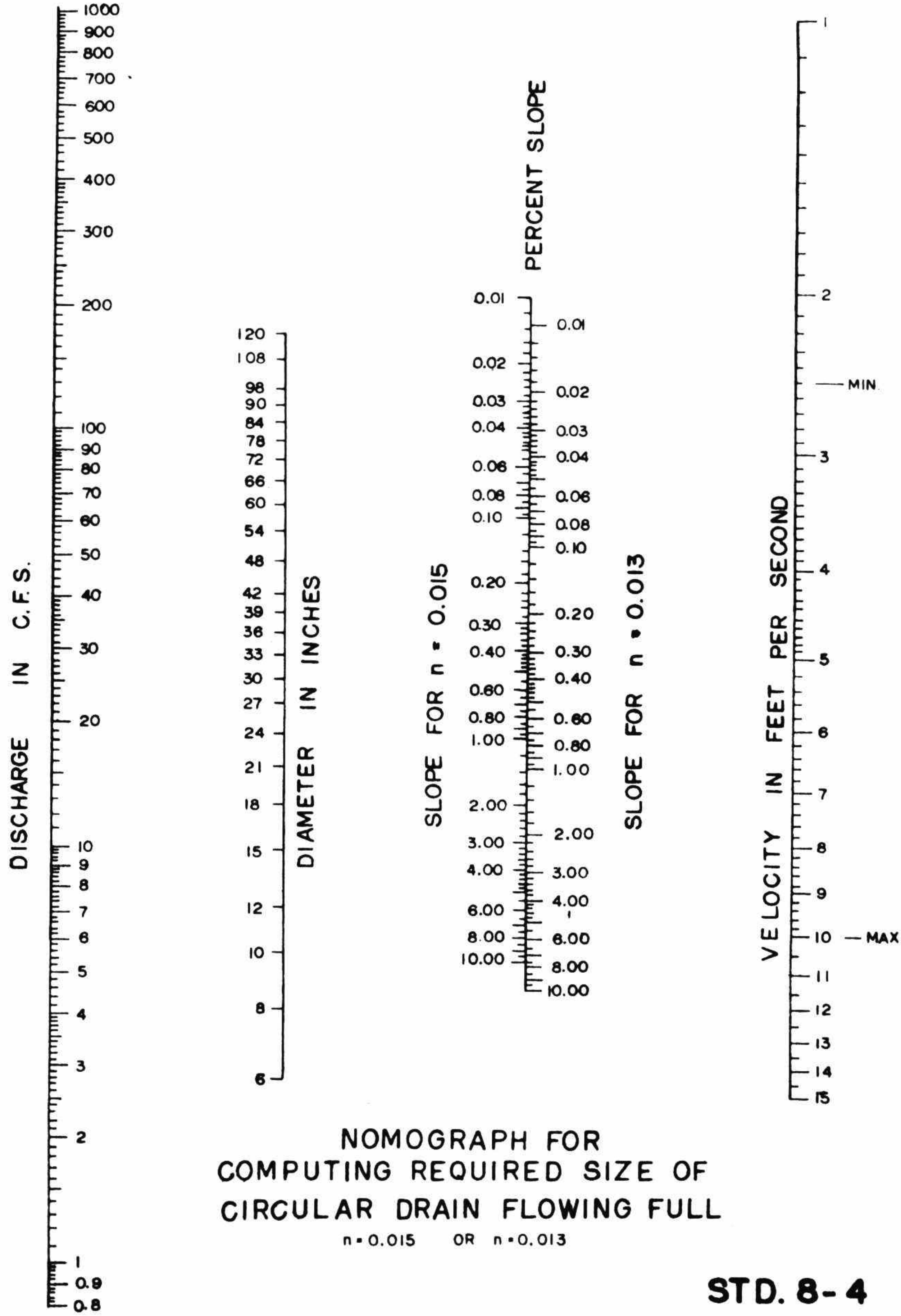
LOCATION	FROM M.H.	TO M.H.	AREA (ACRES)	RUNOFF COEFF	A x R	ACCUM A x R	T OF C (MINUTES)	INTENSITY (IN/HR)	EXPECTED FLOW (C.F.S.)	INVERT UPPER M.H.	ELEVATION LOWER M.H.	FALL	LENGTH	GRADIENT %	PIPE SIZE (INCHES)	CAPACITY (C.F.S.)	VELOCITY (F.P.S.)	TIME OF FLOW (MINUTES)	[illegible] LOWER M.H.
East of Dunbar Road	-	1	12.0	0.5	6.0														
Dunbar Road	1	2	1.5	0.8	1.2														
" "	1	2	2.0	0.5	1.0	8.2	15.0	3.77	30.91				206	0.6	30	31.77	6.47	0.53	
" "	2	3	7.0	0.8	5.6														
" "	2	3	1.5	0.5	0.75	14.55	15.53	3.72	54.13				300	0.68	36	55.0	7.78	0.64	
" "	3	4	8.0	0.8	6.4														
" "	3	4	0.8	0.5	0.4	21.35	16.17	3.65	77.92				265	1.38	36	78.35	11.08	0.40	
" "	4	5	9.0	0.8	7.2														
" "	4	5	2.0	0.5	1.0	29.55	16.57	3.61	106.67				300	1.14	42	107.42	11.16	0.45	
" "	5	6	7.0	0.8	5.6														
" "	5	6	1.3	0.5	0.65	35.80	17.02	3.57	127.80				240	0.8	48	128.47	10.22	0.39	
" "	6	7	9.3	0.8	7.44														
" "	6	7	1.3	0.5	0.65	43.89	17.41	3.53	154.93				340	1.18	48	156.0	12.41	0.46	
" "	7	8	5.7	0.8	4.56	48.45	17.87	3.50	169.75				340	0.76	54	171.4	10.78	0.53	
Dunbar Road	8	9	2.5	0.8	2.0														
Dunbar Road	8	9	2.1	0.8	1.68	52.13	18.4	3.45	179.85				345	0.84	54	180.23	11.33	0.51	
Highway #24	exist	9	19.4	0.8	15.52	15.52		3.45	53.54				270		54				
Dunbar Road	9	out-let	361	0.4	1.44	69.09	18.91	3.41	235.60				821	0.32	72	239.56	8.47		

DRAWN BY I. SUPP. JULY 9, 1971



t	i	t	i	t	i	t	i
0-5	5.50	31	2.67	57	1.77	83	1.32
6	5.28	32	2.62	58	1.74	84	1.31
7	5.07	33	2.57	59	1.72	85	1.29
8	4.87	34	2.52	60	1.70	86	1.28
9	4.68	35	2.47	61	1.68	87	1.27
10	4.50	36	2.43	62	1.66	88	1.26
11	4.32	37	2.39	63	1.64	89	1.25
12	4.16	38	2.34	64	1.62	90	1.24
13	4.00	39	2.30	65	1.60	91	1.23
14	3.88	40	2.27	66	1.58	92	1.21
15	3.77	41	2.23	67	1.56	93	1.20
16	3.67	42	2.19	68	1.54	94	1.19
17	3.57	43	2.16	69	1.53	95	1.18
18	3.48	44	2.13	70	1.51	96	1.17
19	3.40	45	2.09	71	1.49	97	1.16
20	3.33	46	2.06	72	1.48	98	1.15
21	3.26	47	2.03	73	1.46	99	1.14
22	3.19	48	2.00	74	1.45	100	1.13
23	3.11	49	1.97	75	1.43	101	1.12
24	3.05	50	1.94	76	1.42	102	1.11
25	3.00	51	1.92	77	1.40	103	1.11
26	2.96	52	1.89	78	1.39	104	1.10
27	2.89	53	1.86	79	1.37	105	1.09
28	2.83	54	1.84	80	1.36	106	
29	2.78	55	1.81	81	1.35	107	
30	2.72	56	1.79	82	1.33	108	

- 122 -



NOMOGRAPH FOR
COMPUTING REQUIRED SIZE OF
CIRCULAR DRAIN FLOWING FULL
n = 0.015 OR n = 0.013

STD. 8-4

ADAPTED FROM SEELYE - DESIGN, VOLUME ONE PAGE 5-30

PART II - SANITARY SEWERS

INTRODUCTION

Sewage is defined as a combination of (a) the liquid wastes conducted away from residences, business buildings and institutions; and (b) the liquid wastes from industrial establishments with (c) such ground, surface and storm water as may be admitted to or find its way into the sanitary sewers.

The flow rates of sewage, for which sewer capacity should be provided, must be determined from careful considerations of the present and probable future quantities of domestic sewage, commercial and industrial wastes, ground-water infiltration, and any other unavoidable contributions.

A sanitary sewer has two main functions; namely (a) to carry the peak discharge for which it is designed and (b) to transport suspended solids so that deposits in the sewer are kept to a minimum. It is essential therefore, that the sewers have adequate capacity for the peak flow and that it functions at minimum flows without nuisance.

DESIGN FLOWS

One of the first considerations in estimating the sewage flow is to establish a "design period", or a length of time for which the proposed sewers will adequately serve the expected population. Sanitary sewage and industrial wastes are directly related to water consumption and population, so the sewers should be designed for the saturation density of population expected in the design area.

The most common design periods used by Canadian municipalities is a 25 or 50 year period.

Methods of predicting population growth are numerous and include:

(1) arithmetic progression,

(2) constant - percentage growth rate,

(3) graphical comparison with growth rates of similar but larger cities,

(4) graphical extension of past records into the future.

The fourth method is the one most commonly used by municipalities. In many cases, where 'official plans' have been developed, the proposed zoning and therefore predicted population densities will act as a guide for the sewer designer.

A list of population densities commonly used in sewer design is shown below.

Area Type	No. of Persons per Acre
Single Family	20 - 35
Semi-detached	30 - 50
Multi-family, row housing	60 - 100
Apartments	400 - 600
Commercial	20 - 30

Sanitary sewage flow varies throughout the day, week, season and according to the water consumption.

Sanitary sewer design is almost always based on the peak flow rate expected and then the flow velocities are checked at the average and minimum flow rates to determine whether settling will occur or not.

As a rule of thumb, main sewers are designed for 250% of the average flow rate and lateral sewers are designed for 300-500% of the average rate.

Domestic water consumption varies from 50 - 250 g.p.c.d., with an average use of 120 - 150 g.p.c.d. Sewage flow is estimated to be 70% of this water consumption; the other 30% being used for lawn watering, car-washing, street cleaning, etc. The average daily sewage flow being used by most municipalities is 100 g.p.c.d.

Values that are often used are:

(1) max. daily flow rate = 2 x average daily flow,
(2) max. hourly flow rate = 3 x average daily flow,
(3) min. daily flow rate = 2/3 x average daily flow,
(4) min. hourly flow rate = 1/3 x average daily flow.

Formulas often used for calculating extremes are:

(1) $Q\ max./Q\ avg. = 5.0/p^{1/6}$

(2) $Q\ min./Q\ avg. = 0.2/p^{1/6}$

(3) $Q\ max./Q\ min. = 25.0/p^{1/3}$

where p = population in thousands.

No peaking factor is normally applied to infiltration (usually 0.002 - 0.004 c.f.s./ac.) and reduced peaking factors are usually applied to industrial contributions, since industrial peaks can be predicted with greater accuracy.

In the actual design of sanitary sewers after the capacity required is arrived at, there are several considerations in pipe size, gradient, material which must be considered. The flow cannot be as slow at low flows as to allow the solids and sediment to settle out in the bottom of the pipe. The flow cannot be of such a speed that the solids sediment and grit scour or wear away the pipe. There is also the problem of the formation of Sulphide Gases if the flow is too slow. This applies particularly to large trunks. This gas will deteriorate concrete, steel and cast-iron pipes.

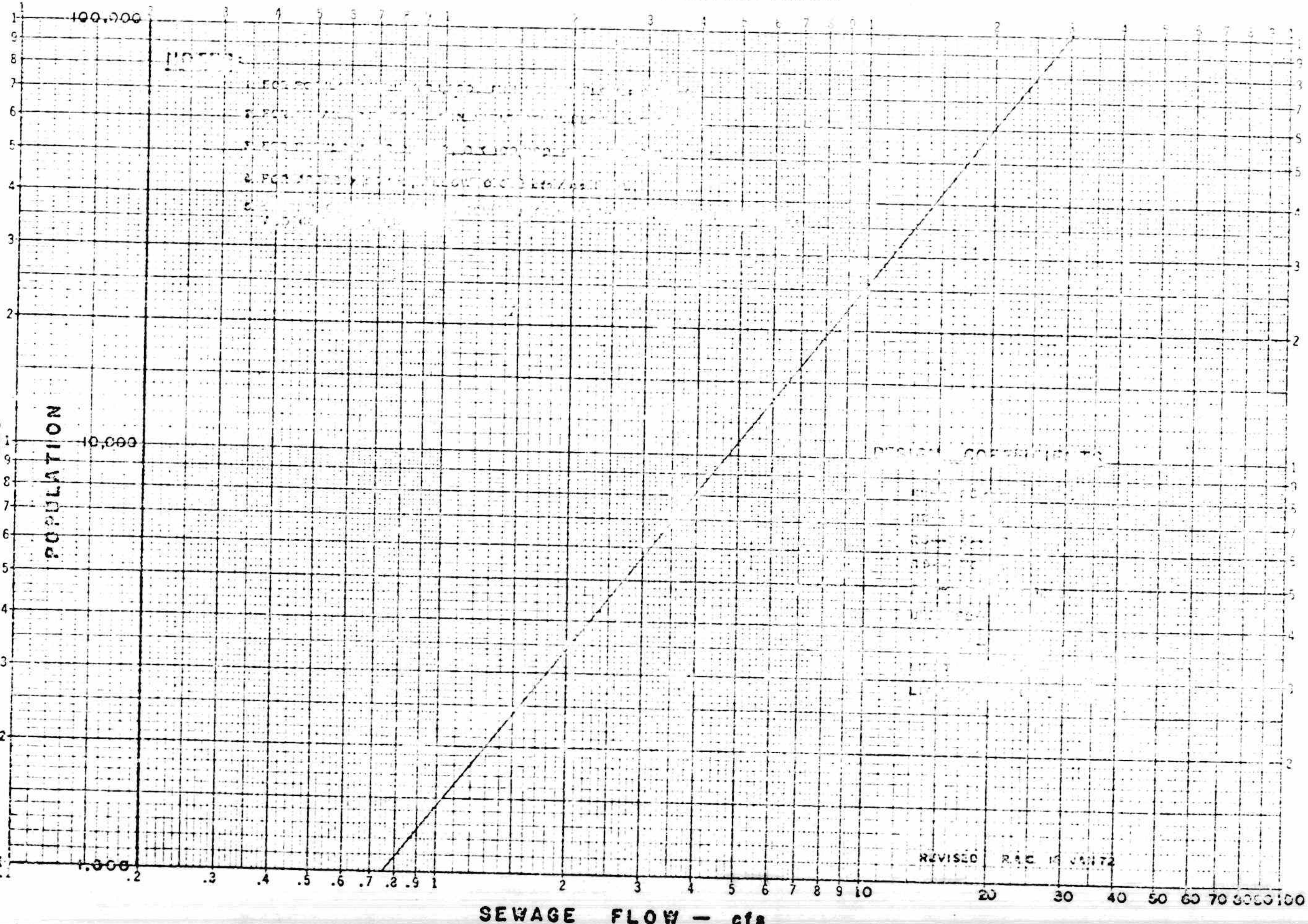
A sewer pipe must be designed for the depth of trench, traffic load, railway loading or any special application.

One of the other lecturers has already or will, in a later session, cover the structural aspects of sewer design.

I have included here a copy of our present sanitary sewer design curve and our design sheet. As you will notice our curve is based on population and a direct reading for expected sewage flow. Many municipalities use a curve based on acreage and zoning, but it always comes back to the population, since people create sewage.

Also attached is a copy of our design criteria which is used for new subdivisions in Galt. I won't go through this since these are different for every municipality and no doubt many of you will either agree or disagree with many of the requirements.

CITY OF GALT
SANITARY SEWER DESIGN FLOWS



SANITARY SEWER DESIGN CHART

STD. 9-1 (A)

DRAINAGE AREA PLAN NO

PROJECT NO

n =

DESIGNED BY

DATE

LOCATION	FROM M.H.	TO M.H.	AREA (ACRES)	DENSITY PERSONS PER ARCE	POPULATION	CUM AREA (ACRES)	CUM POPULATION	SEWAGE FLOW (C.F.S.)	INFILTRATION FLOW (C.F.S.)	TOTAL FLOW (C.F.S.)	LENGTH	GRADIENT (%)	PIPE SIZE (INCHES)	CAPACITY (C.F.S.)	VELOCITY (F.P.S.)	DROP IN LOWER M.H.

DATE: JULY 12, 1971 DRAWN BY J. CURRIE

PHYSICAL FEATURES OF SEWER SYSTEMS

INTRODUCTION

The discussion on physical features of sewer system will involve a look at various appurtenances essential to a properly designed sewer and a brief review of the most commonly used sewer materials.

APPURTENANCES

One of the most important and most common structure found in both storm and sanitary sewers is the manhole. Customarily, a manhole is required at all changes in grade, pipe size, direction of flow and quantity of flow.

The size of the manhole is usually determined by the size of the largest pipe in the manhole, but seldom are they less than 4 foot diameter (for pre-cast) or 4 foot square for poured in place manholes. Their principal purpose is to permit the inspection and cleaning of the lines and the removal of obstructions. In most municipalities, the allowable spacing of manholes is 300-350 feet for sewers up to 43"Ø, when cleaning is done by rodding or flushing and up to 500-600 feet for sewers over 48" where the lines may be easily entered by maintenance crews. The interior base of the manhole is always formed with mortar or concrete to a "U" shaped channel to allow smooth, continuous flow and to minimize the head losses at these points.

For storm sewers, probably the most important structure is the catch basin or storm inlet. Regardless of the adequacy of the underground sewer system, proper drainage cannot result unless storm water is quickly and efficiently collected and introduced into the system. No specific inlet type can be considered best for all conditions of use. Street grades, cross-slopes and depression geometery all affect the hydraulic efficiency. The major inlet structures are usually one of the following:

(1) gutter inlet - a standard C.B. sitting in a slight depression in the gutter,

(2) arch back inlet - a C.B. inlet composed of both curb and gutter openings acting as an integrated unit.

(3) a ditch inlet - an oversized C.B. with its frame usually inclined at 30 - 45°.

Catch basins are usually constructed with a 12" - 18" sump in the bottom which collects grit, leaves and other debris which would normally enter the sewer system and must be cleaned on a regular routine basis, twice yearly is a minimum recommendation.

House connections or laterals play an important role in sanitary sewer design and should never be taken for granted. Most municipalities accept a 4"Ø lateral as the absolute minimum size with some using 5" or 6" diameter connections. These laterals, which should be laid to a grade of not less than 1/8in./ft. (1%) are usually connected to the main sewer with Tee's, Wye's or Saddles.

The use of a Tee is preferable in sanitary sewers since experience has indicated frequent breakage of the Wye where rodding of the house connection has become necessary.

Another sewer appurtenance, which is not seen very often is called an inverted syphon. This refers really to a depressed

sewer which flows under a head, i.e. under pressure and carries the flow under some obstruction such as a river or a highway.

I have attached copies of several standard drawings for appurtenances presently used in Galt which you may look at.

We're represented here by many municipalities and I think you'll agree that there are at least that many different standard drawings for catch basins and manholes. If you have any questions on these drawings please mention them.

MATERIALS USED IN SEWER CONSTRUCTION

When the designer is deciding on what type of sewer material he will call for in the tender specifications, he is always concerned with one or more of the following factors:

(1) flow characteristics - friction factor,

(2) life expectancy,

(3) resistance to scour,

(4) resistance to acids, alkalis, solvents,

(5) ease of handling and installation,

(6) structural strength,

(7) type of joint,

(8) availability of sizes.

No one material meets all these requirements. The three most commonly used materials are concrete (plain or reinforced), asbestos-cement and vitrified clay.

We will now compare these three materials.

A. ASBESTOS-CEMENT - this pipe is available in sizes 4" - 36" diameter and can be used in gravity and pressure lines. Jointing is done using a confined 'O' rubber ring gasket. This pipe is extremely light and easy to handle and the longer laying lengths (13') mean fewer joints and thus less problem of infiltration. The large lengths also enable the contractor to lay the pipes to truer line and grade. This type of pipe is subject to corrosion by acids and septic sewage in sanitary lines and can be easily eroded by grit in high velocity (steep grade) storm lines.

B. CONCRETE PIPE - unreinforced concrete pipe is available in sizes 4" - 24"Ø and in sizes 8" - 120"Ø for reinforced pipe.

Concrete pipe is used primarily for gravity sewers, although reinforced concrete pressure pipe and prestressed concrete pressure pipe are available for force mains, submerged outfalls and inverted syphons. A variety of joints are available from rubber gasket to plain mortar depending on the water-tightness demanded by the existing ground conditions.

Concrete pipe can be produced in varying strengths and offers the best structural supporting characteristics of the three materials. This pipe is also subject to corrosion where acids are present in the sewer, where velocities are not sufficient to prevent septic conditions (i.e. the formulation of hydrogen sulphide) and where the soils are highly acid or highly sulphate alkaline.

C. VITRIFIED CLAY PIPE - this pipe is readily available in sizes 4" - 36"Ø and larger in some areas. Vitrified pipe is manufactured in both standard strength and extra strength for load support and is available with a mechanical joint on the new flex-lox joint which is similar to bell and spigot pipe.

The main advantage of vitrified pipe over the others is that it is highly resistant to corrosion from most acids.

CONSTRUCTION

A very important part of the overall considerations is the construction of the sewers. There is not time to go into lengthy detail, but I will try to cover some general points. To operate efficiently and avoid unnecessary maintenance and general operating problems, sewers must be laid to line and grade and remain on line and grade. This is usually accomplished with grade stakes to which batter boards and string lines are set. Some contractors, particularly on larger pipes will use a transit to keep the pipe on line. Today a Laiser Beam is available which is set in the downstream manhole and projects through the pipe to a target placed in the bell of the pipe being set. This provides for very rapid progress of the work and contractors who have used them claim at least twice the daily production.

The bedding of the pipe is of equal importance to ensure the pipe does not settle after backfilling. This also includes the compaction of good granular material around and at least 12" above the pipe to assure that first, the sides of the pipe are supported to the trench wall so that its structure can carry the load on it, that it will not be displaced out of line and last, that it will not be damaged during the backfilling operations. When completed a good job should look like a gun barrel and under no circumstances should less than 2/3 of the pipe be visible.

Other points an inspector should be watching for are:

(1) the bridging of the pipe from bell to bell on the bedding, the bedding must be to grade and a hole for the bell dug out.

(2) placing of pieces of wood under the bell or front edge to adjust to grade.

(3) Poor or no compaction effort under, around and over the pipe.

(4) Large boulders in the backfill even with some cover on the pipe.

(5) Concrete under inlet outlet pipes between base of manhole and natural excavated ground. This also applies to catch basin leads.

(6) Proper connection of house services at the main and a proper plug in the end, especially where connecting to an older sewer. Workers or contractor will place the connecting pipe into the main causing blockages, bucket machines to bang up and perhaps rip out the pipe and possibly some of the main. They also block the passage of T.V. cameras making full inspection impossible.

(7) A very important duty of the inspector is to ensure that the joints are made properly whether they are hemp and mortar, refined 'O' ring or collar type that the bell and spigots are in good shape, not damaged, and are perfectly round and fit properly.

(8) Smaller pipe are usually put home by hand with a large bar. They must be fully put home and not spring back. Larger pipe should be put home with a come-along operated from inside the pipe. In no case should the backhoe bucket be used to put pipe home. I have seen this done successfully, but there is danger of damaging a joint several lengths back which may be backfilled and not detected.

DESIGN CRITERIA FOR MUNICIPAL SERVICES

IN

THE CITY OF GALT

A. SANITARY SEWERS

1. Pipe Sizing - minimum size 8" diameter.

Design calculations for sanitary sewer systems shall be completed on City of Galt "Sanitary Sewer Design Charts" (Std. 9-1(A)).

a) Design Flow -

Refer to drawing Std. 9-2 based on the following densities:

R1	-	25 persons per acre
R2	-	35 persons per acre
R3 and R4	-	45 persons per acre
Unzoned	-	40 persons per acre
Schools	-	35 persons per acre
Industrial	-	0.032 c.f.s. per acre
Commercial	-	0.032 c.f.s. per acre

For infiltration add 0.002 c.f.s. per acre to the above figures.

b) Pipe Capacity -

For determining pipe capacity, use Manning's Formula (City of Galt standard circular drain nomograph Std. 8-4)

Use: n = 0.013 for asbestos cement, vitrified clay and concrete pipe.

c) Flow Velocities -

Maximum 12.5 feet per second, pipe flowing full.
Minimum 2.5 feet per second, at actual flow.

d) Selection of Bedding and Class of Pipe -

Refer to C.C.P.A. Design Manual for pipe class and bedding requirements for standard designs. Special designs shall be completed on design chart Std. 8-3 in accordance with the following:

For calculating transmitted live loads on sewer pipes, use Marston's Formula:

$$Wt = \frac{1.0}{A} \times Ic \times Ct \times T$$

in which:

Wt = average load per unit length of pipe (pounds per lineal foot)

A = length of pipe on which the load is computer (feet)

Ic = impact factor for a moving load

Ct = load coefficient

T = a concentrated surface load (pounds)

A. SANITARY SEWERS

1. Pipe Sizing (continued)

For vehicular traffic use H-20 loading.

For railroad traffic use E-70 loading.

For calculating backfill loading on sewer pipes use Marston's Formula:

$Wd = Cd \times W \times Bd^2$ for trench conditions and

$Wc = Cc \times W \times Bc^2$ for embankment conditions

in which:

Wd & Wc = the soil backfill load in pounds per lineal foot of pipe length.

Cd & Cc = the load calculation formula coefficient

W = the unit weight of the backfill material in pounds per cubic foot.

Bd = the width of trench in feet, measured in a horizontal plane at the extreme top of the pipe.

Bc = the O.D. of the pipe in feet.

For non-reinforced pipe multiply above loading by safety factor of 1.25.

Use minimum A.S.T.M. 0.01" crack, three-edge bearing strength requirements for reinforced concrete pipe and minimum A.S.T.M. three-edge bearing crushing strength requirements for non-reinforced concrete pipe.

Refer to drawing Std. 3-1 for standard pipe beddings.

e) Pipe Depth and Location-

A minimum cover of eight (8) feet, from proposed road grade, is required to the top outside edge of the pipe barrel.

The sanitary sewer shall be located five (5) feet North and East of centreline of the road allowance, unless approved otherwise.

f) Type of Pipe and Joint Acceptable for Sanitary Sewers -

(i) Vitrified clay - 'flex-lox' joint pipe, extra strength only.

(ii) Asbestos cement- with confined 'O' rubber gaskets, in residential areas only.

(iii) Concrete - with acceptable rubber gasket, in residential areas only.

All concrete sewer pipe up to and including 18" diameter shall be equal to A.S.T.M. C14/68 or latest amendment.

All concrete sewer pipe over 18" diameter shall be equal to A.S.T.M. C76/68T or latest amendment.

All asbestos cement pipe shall be equal to A.S.T.M. C-428-63T or latest amendment.

All vitrified clay pipe shall be equal to C.S.A. A60 or latest amendment.

2. MANHOLES

a) Poured or precast, as per City of Galt Standards.

b) Manholes shall be located at a maximum distance of 300 feet apart unless otherwise approved.

c) A drop inlet manhole will be required if a drop in excess of 3'-0" occurs between any invert and the lowest invert in the manhole.

d) The benching in all manholes must not be less than 9" in width and shall conform to the City of Galt Standard Drawings.

e) A special detail must be shown for all manholes when:

(i) benching differs from that shown on the standard drawings.

(ii) depth or shape of manhole requires additional reinforcing.

(iii) a situation occurs that is not shown on the Standard Drawing.

f) All manholes must be referred to a City standard on the profile above the centreline of the road profile at each manhole.

g) All manholes shown in the profile must indicate all existing and proposed inverts with each having reference to the North arrow.

3. CONNECTIONS-

A single sanitary connection shall be provided for each dwelling in the subdivision. All sanitary sewer connections will be of asbestos cement pipe with rubber ring joints. In general, house connections should be shown to the centre of each lot.

Sewer connections shall be made with properly designed fittings in accordance with the Standards of the City of Galt

Two inch by four inch (2" x 4") wooden markers from invert to two feet (2') above ground level be placed at the end of each connection, the top two feet (2') to be painted red.

NOTE:

i) Sanitary sewers are not permitted to accept roof drainage, or drainage from internal storm systems.

ii) Vitrified clay pipe shall be used when industrial acreage is considered in the design of the sanitary sewer.

iii) All existing basement elevations must be shown on the profiles.

iv) Sanitary house connections shall connect directly into a manhole wherever practicable.

B. STORM SEWERS

1. Pipe Sizing -

The minimum diameter for mains and double catch basin connections shall be 12 inches and for single catch basin connections, 10 inches.

B. STORM SEWERS

1. Pipe Sizing - (continued)

Design calculation for storm sewer systems shall be completed on City of Galt "Storm Sewer Design Charts" (Std. 8-1(A)).

a) Run-Off

To calculate flow, the rational method (Q=AIR) shall be used where:

Q	=	design flow in c.f.s.
A	=	area in acres
I	=	rainfall intensity in inches per hour
R	=	run-off coefficient

I is to be determined from the City of Galt standard rainfall intensity curve, Std. 8-2. R is to be determined from the City of Galt standard storm sewer analysis chart, Std. 8-1.

Storm sewers are to be designed to accommodate flow from all areas, which in the opinion of the City Engineer require outfall through the subdivision.

Storm sewer connections, properly designed, will be required for all commercial, institutional and industrial lots or blocks within the plan of subdivision.

b) Pipe Capacity -

For determining pipe capacity use Manning's Formula (City of Galt standard circular drain nomograph Std. 8-4).

Use:

n = 0.013 for vitrified clay, asbestos cement and concrete pipe.

n = for corrugated metal pipe must be taken from the A.I.S.I. Handbook.

c) Flow Velocities -

Maximum 15 feet per second, pipe flowing full.

Minimum 2.5 feet per second, pipe flowing full.

d) Selection of Bedding and Class of Pipe -

Refer to Section A, Item 1(d).

e) Pipe Depth and Location -

A minimum cover of five (5) feet, from proposed road grade is required to the top outside edge of the pipe barrel. The storm sewer shall be located five (5) feet South and West of centreline of the road allowance unless approved otherwise.

f) Type of Pipe and Joint Acceptable for Storm Sewers -

(i) Concrete - with acceptable rubber gaskets unless otherwise approved by the City Engineer.

(ii) Corrugated Metal - for culverts only.

B. STORM SEWERS

1. Pipe Sizing - (continued)

All 10" concrete sewer pipe shall be equal to A.S.T.M. C14/68 (extra strength only) or latest amendment.

All concrete sewer pipe 12" diameter and larger shall be equal to A.S.T.M. C76/68T or latest amendment.

2. Manholes and Catch Basins

a) Poured or precast as per City of Galt standards.

b) Manhole and catch basin spacing shall not exceed 300 feet unless otherwise approved.

c) A drop inlet manhole will be required if a drop in excess of 3'-0" occurs between any invert and the lowest invert in the manhole.

d) The benching in all manholes must not be less than 9" in width and shall conform to the City of Galt standard drawings.

e) A detail must be shown for all manholes when:

i) benching differs from that shown on the standard drawings.
ii) depth or shape of manhole requires additional reinforcing.
iii) a situation occurs that is not shown on the Standard Drawings.

f) All manholes must be referred to a City standard on the profile above the centreline of the road profile at each manhole.

g) All manholes shown in the profile must indicate all existing and proposed inverts with each having reference to the North arrow.

h) When road grades are 5.0% or greater, the maximum spacing between catch basins shall be reducted to 200 feet.

i) Where road grades exceed 5.0%, catch basins shall be of the offset type, and constructed in accordance with Std. 2-9.

3. Backlot Drainage

Lots shall be graded from back to front wherever possible. All drainage ditches and swales shall be sodded in accordance with the current City of Galt specifications.

Rear lot catch basins shall not be employed unless previously approved by the City Engineer.

NOTES:

i) All storm sewers and appurtenances shall be designed and constructed in accordance with the most recently revised City of Galt standard drawings and specifications.

ii) The Owner shall grade all major rear yard and side yard swales, as shown on the grading plan, to provide proper drainage. Such works shall be performed prior to performance acceptance of the underground utilities.

B. STORM SEWERS

3. Backlot Drainage - (continued)

iii) A minimum clearance of 9" is required between outside pipe surfaces at all pipe crossings.

iv) No radius pipe shall be used for pipe having a diameter of less than 21".

v) A double catch basin is required where drainage is received from more than one direction.

C. ROADS

1. Roadways -

All geometric design criteria with respect to horizontal and vertical control elements must conform with the "Geometric Design Standards for Canadian Roads and Streets" as published by the Canadian Good Roads Association.

2. Intersections -

All radii for curbs to be 25' in length for 30 foot pavements unless approved otherwise. For intersections of other pavement widths, the curb radii are as follows:

Width of Intersection Pavement (feet)	Curb Radius (feet)
30 and 36	30
30 and 48	50
36 and 36	30
36 and 48	50
48 and 48	50

3. Driveway Entrances -

Driveway entrances and curb cuts shall be in accordance with the standard drawings. Special designs will be required for commercial and industrial driveways depending on expected use.

4. Pavement -

All roadways shall be designed by the Consulting Engineer but in no case will the design be less than:

i) 4" - granular "B" material, or equivalent

ii) 4" - granular "A" material

iii) 2" - binder coarse asphalt

iv) 1" - surface coarse asphalt

(Asphalt shall be one of Groups I, II and III, depending on the classification of the proposed road, which shall be determined by the City Engineer.)

C. ROADS

5. General -

i) The maximum allowable road grade for all roads is 6.0%, unless approved otherwise. The minimum allowable road grade is 0.50%, except for cul-de-sacs and turning circles where a curb grade of 0.50% must be maintained.

ii) No change in grade greater than 1.0% is allowed without a vertical curve. The minimum length of each grade is 50 feet.

iii) The allowable crowning of the road shall be 2.5% - 3.0%.

iv) All existing and proposed services, curbs and sidewalks must be dimensioned within the street line.

D. CURBS

Curb sections are shown on the standard plans and the developer is required to provide one standard driveway entrance and concrete ramp at the required location for each dwelling unit.

All materials must meet the requirements as set out in the curb specifications and standard drawings.

A thoroughly consolidated base of crushed granular material will be required underneath and 6" beyond each side of the curb section as indicated on the standard cross-sections.

E. SIDEWALKS

Sidewalks shall be constructed in accordance with the sections shown in the standard drawings. Concrete shall be poured to a depth of 8" where the sidewalk intercepts a commercial or industrial driveway, for the full width of the driveway.

In general, sidewalk construction must conform to the current City of Galt standard specifications for this work.

F. BOULEVARDS

All portions of the road allowance between the back of curb and the edge of sidewalk closest to the curb shall be sodded with No. 1 Nursery Sod including 4" of topsoil.

G. TREE PLANTING

The Owner shall pay for the planting of such species of trees and in the locations as may be approved by the Public Utilities Commission of the City of Galt. These trees shall be planted as follows:

- one tree per dwelling unit unless there is already a tree on the boulevard.
- a minimum of 40 feet apart.

The Owner shall guarantee the life of such trees during the first year of planting.

CITY OF GALT ENGINEERING DEPARTMENT

ENGINEERING SUBMISSIONS FOR RESIDENTIAL DEVELOPMENT

GENERAL

The following requirements cover submissions to the City Engineering Department only. Additional submissions to the Public Utilities Commission of the City of Galt shall be made directly to them in accordance with their requirements. The second submission shall not be made unless the Public Utilities Commission's comments regarding the first submission have been received and incorporated.

Plan prints for all submissions shall be rolled and placed in numerical order. All prints shall be stamped with the submission number (1, 2, etc.) and date of submission.

The original drawings shall be on linen or other transparent material as may be approved by the City Engineer.

All drawings shall measure 24" x 36" or 24" x 48".

NOTE:

All plans shall show the following note:

- Single lots shown as 37
- Semi-detached lots shown as 69

GENERAL PLANS

A. All General Plans shall:

1. Be drawn at a scale of 1" = 100'.
2. Show a key plan at a scale of 1" = 1000'.
3. Show a North arrow.
4. Show all existing and proposed lot numbers and blocks.
5. Refer all datum to a Standard City of Galt Geodetic Bench Mark.
6. Show all existing services and utilities and abutting property limits in broken lines.
7. Show the standard City of Galt title block. (Std. 11-5)

B. General Plans showing above-ground services shall:

1. Show all existing and proposed curbs, road allowances, street names, catch basins and road grades.

C. General Plans showing below-ground services shall:

1. Show all existing and proposed sewer lengths, sizes, types, manholes, grades (to two decimal points) and direction of flow.
2. Show all existing and proposed watermains, sizes, hydrants, valves and fittings.
3. Show all house connections (as singles).

GENERAL PLANS (continued)

D. General Plans showing drainage shall:

1. Show storm drainage areas, acreage, and run-off coefficients.
2. Show sanitary drainage areas, acreages and densities.
3. Show existing elevations and proposed elevations and other pertinent information, and swale locations, together with specific elevations at the following points:

   i) centreline of each lot at 20' setback.
   ii) centreline of each lot at 60' setback.
   iii) both corners at the rear of each lot
   iv) centreline of the proposed road in front of each lot.
   v) existing and proposed elevations around the limits of the subdivision and existing elevations extending at least 200' externally.
   vi) both corners at the front of each lot.

4. Show all berms by heights and locations.
5. Show cross-sections of all proposed swales.
6. Show all slopes created indicating top of slope and slope ratio of the slopes.
7. Show in a Legend a sample lot indicating rear lot line, existing and proposed elevations at rear lot corners, lot number, existing and proposed elevation at 20' setback, proposed elevation at 60' setback, existing and proposed front lot corners, street line, proposed centreline road grade and direction of flow by arrows.
8. Show if there is a design split in the grading of the lot.
9. The proposed elevation at 20' setback from the street line should be a minimum of 12" and a maximum of 36" above centreline of road in front of the lot, unless approved otherwise.

PLANS AND PROFILES

1. All plans and profiles shall be drawn at scales of:

   1" = 40' horizontally, and

   1" = 4' (or 1" = 10') vertically.

2. Show the North arrow in each plan view.
3. Show the standard City of Galt title block. (Std. 11-6)
4. Where two or more sheets are required for each street, match lines must be used and there should be no overlaps or duplication of information.
5. Two short streets may be shown on one plan profile if space permits.
6. Refer datum to a standard City of Galt Geodetic Bench Mark, stating Bench Mark number, location and elevation.
7. Show all existing and proposed curbs, road allowances and street names.
8. Show all existing lot numbers and blocks, all existing iron bars and registered plan numbers.

GENERAL PLANS (continued)

PLANS AND PROFILES (continued)

9. Show all existing and proposed watermain sizes, with hydrants, valves, fittings and other utilities.
10. Show all existing and proposed sewer lengths, sizes, types grades (to two decimal points), direction of flow, manholes and catch basins.
11. Show all house connections (as singles).
12. Show all manholes with proper symbols and their types referred to a City of Galt standard.
13. Storm manholes shall be designated as: M.H. 2 Sanitary manholes shall be designated as: M.H. S-4.
14. Centreline of road shall be plotted on the plan view with stations of B.C.'s, E.C.'s, road limits and intersections.
15. On the profile show the length between grade points, the proposed grades at 50 feet intervals and all vertical curve information.
16. Show the street line elevations at the limit of the subdivision.
17. On all profiles show the type of sewer bedding to be used and the maximum allowable trench widths.
18. Show a cross-section of pavement design indicating depths and types of the granular material and the asphalt.
19. Show a cross-section of all major swales and drainage ditches, if any.
20. Show cross-sections of all walkways.

FIRST SUBMISSION

The following plans and documents are required for First Submission:

A. Two complete sets of the following drawings:

i) Plan proposed for registration, if available.
ii) General Plan showing all above-ground services.
iii) General plan showing all below-ground services.
iv) Overall grading plan showing all existing and proposed elevations as described in the previous section.
v) Plan and profile of all proposed road and services.
vi) Plan showing miscellaneous details, if required.

B. Two sets of the schedule of services required and cost estimates, including a breakdown of all cost sharing if any.

C. Two copies of the following:

i) A storm sewer drainage plan including the whole storm sewer area to be drained through the proposed system, showing contours and drainage patterns of adjacent lands.
ii) Standard Design Sheet for storm sewers. Std. 8-1(A).
iii) A sanitary sewer drainage plan including the whole sanitary sewer area to be drained through the proposed system.
iv) Standard Sanitary Sewer Design sheet. Std. 9-1(A)
v) Standard design sheet for pipe strength and bedding unless if special designs are required.

GENERAL PLANS (continued)

SECOND SUBMISSION

The following plans and documents are required for the Second Submission:

A. Two complete sets of drawings as listed for first submission, Item A.

B. Two complete sets of schedules as listed for first submission, Item B, including three (3) draft copies of the contract specifications and standard drawings to be used.

C. i) Two complete sets of drawings and sheets as listed for first submission, Item C, if unsatisfactory in first submission.

ii) O.W.R.C. Application Forms for Sewers (original plus two copies) duly signed by the Consulting Engineer, together with

iii) An additional set of plans and profiles of all proposed roads and services and a general plan of underground services, to be submitted by the City to O.W.R.C.

D. Copies of all approvals (Department of Transportation and Communications, H.E.B., C.N.R., etc.) if applicable.

AS CONSTRUCTED SUBMISSION

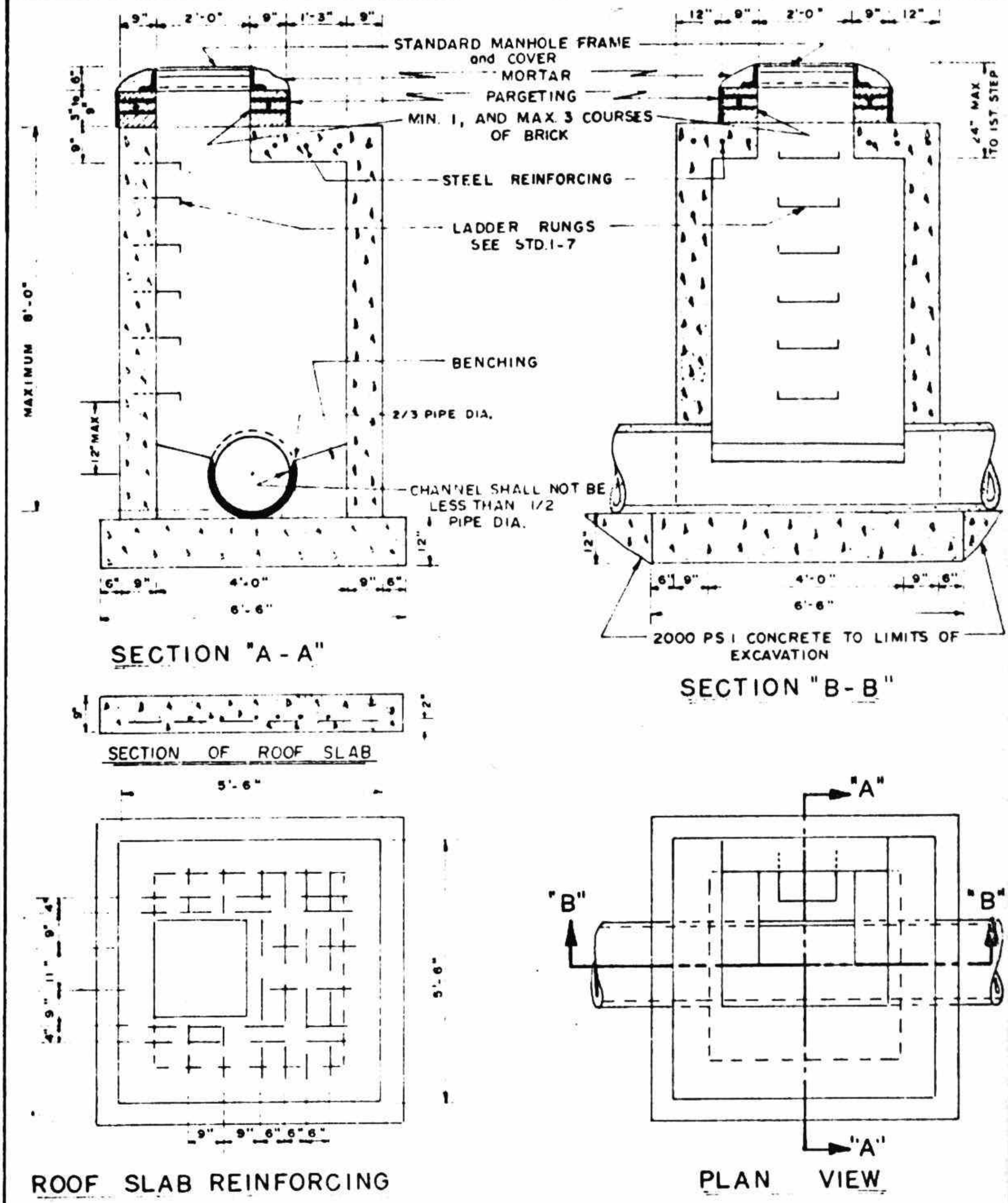
When construction is complete, one set of linen drawings showing "as-built" details and all iron bars is to be submitted to the City Engineer for filing with the Engineering Department of the City of Galt.

NOTES:

1. The above submission must be made and approved before any final approvals can be made.

2. Prior to "performance acceptance" of the underground utilities, the Owner's Engineer shall provide the City with "as-built" sepias of all the work completed. This shall be in addition to the two prints of the "as-builts" which shall accompany the letter to the City requesting a "performance inspection" of any completed works.

3. Where any City cost sharing is involved, the Owner's Engineer shall provide the City with three (3) complete "Schedule of Unit Prices" prior to start of construction.



NOTES:

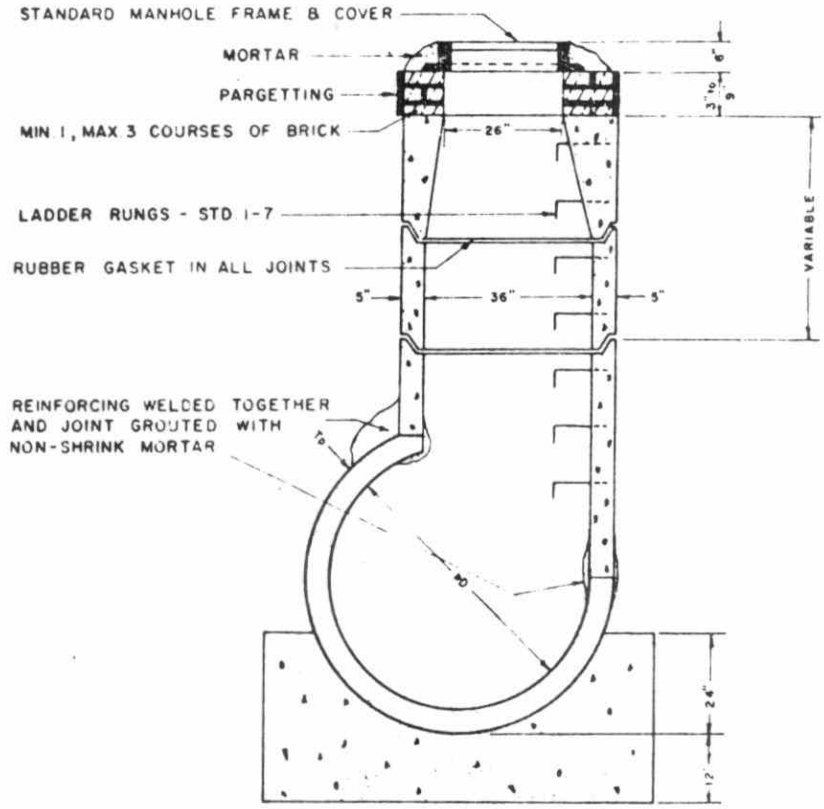
1. CONCRETE TO BE 3000 P.S.I. COMPRESSIVE STRENGTH AT 28 DAYS.
2. MORTAR MIX: 1 CEMENT: 3 SAND.
3. PARGET MIX ON ALL BRICK WORK TO BE 1 CEMENT 3 SAND & APPLIED 1/2" THICK.
4. USE APPROVED CONCRETE BRICK 3000 P.S.I.
5. MANHOLE FRAME & COVER SUPPLIED BY THE CITY.
6. BENCHING SLOPES 1 VERT: 6 HOR.
7. STEPS: FIRST STEP 24" MAX. BELOW FRAME
   LAST STEP 12" MAX. ABOVE BENCHING.
8. COVER OF CONCRETE OVER REINFORCING TO BE 2".
9. ALL REINFORCING TO BE NO. 5 HI-BOND STEEL BARS.
10. ALL BASE SECTIONS TO BE FORMED.

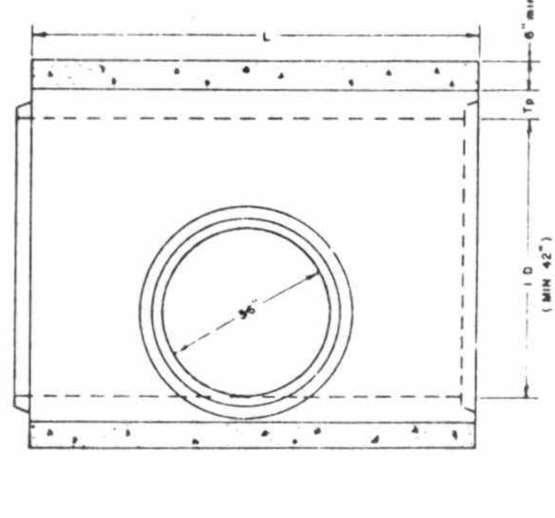
NO.	REVISIONS	BY	CHK'D.	DATE
2	BENCHING	W.B.	D.N.	1/15/69
1.	POSITION OF LADDER RUNGS CHANGED	W.B.	D.NARAINE	JAN 7/69

CITY OF GALT

STANDARD MANHOLE UNDER 8' DEEP for SEWERS 39" or SMALLER

SCALE: 3/8" = 1'-0"	STD. 1-1
DATE: APRIL 10, 1968	
DRAWN BY: W. BALL	P.ENG.
DESIGN BY: P. MORROW	CITY ENGINEER





NOTES:

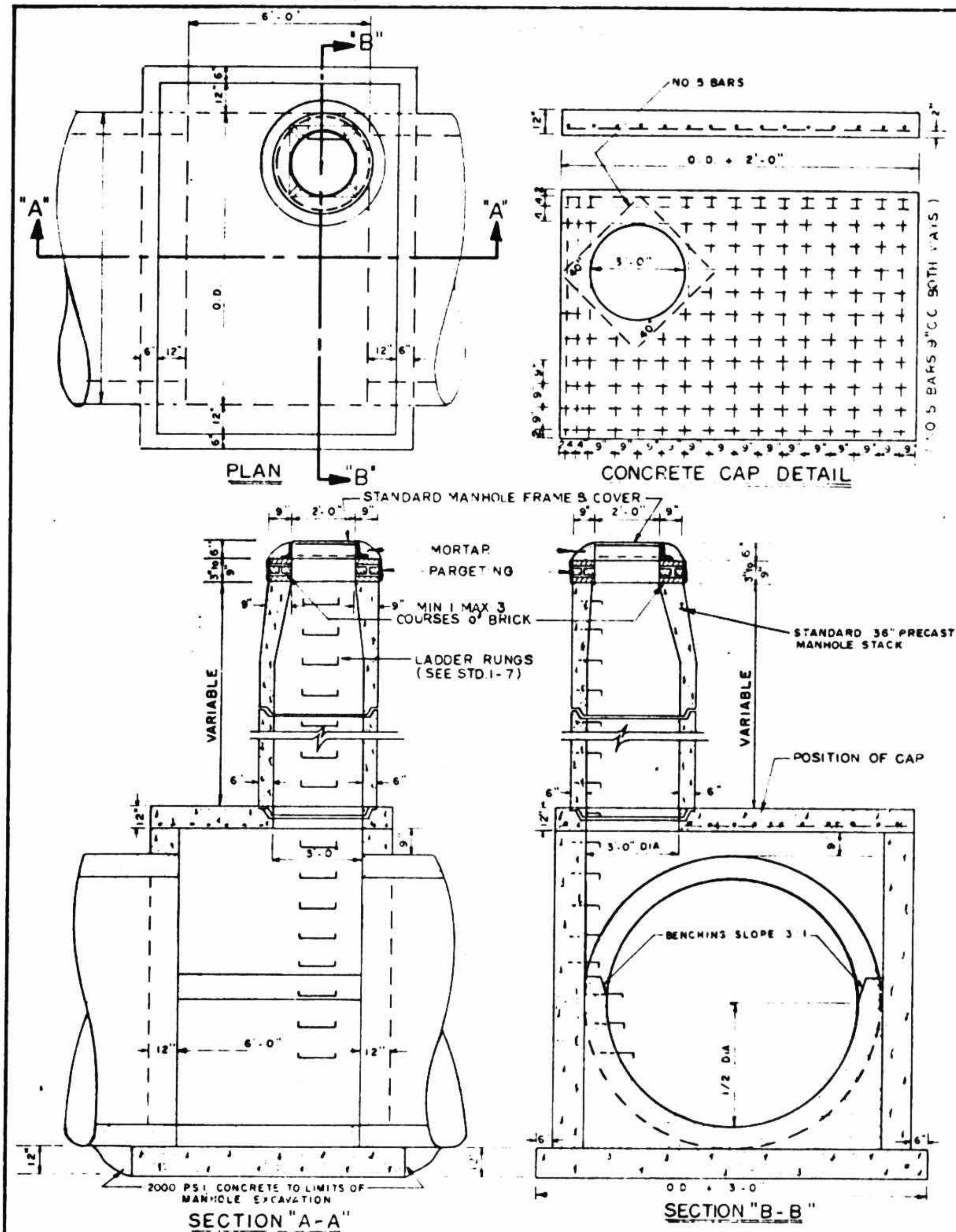
1. MIN. PIPE DIA. OF MAIN LINE TO BE 42" I.D.
2. FOR I.D. < 60" - L = 90".
3. FOR I.D. 60" & LARGER - L = 96".
4. PARGET MIX ON ALL BRICKS TO BE 1 CEMENT : 3 SAND, APPLIED 1/2" THICK.
5. USE APPOVED CONC. BRICK - 3000 P.S.I.
6. M.H. FRAME & COVER SUPPLIED BY CITY.
7. STEPS: FIRST STEP 24" MAXIMUM BELOW FRAME. LAST STEP TO BE A MAXIMUM OF 24" ABOVE PIPE INVERT.
8. CONCRETE TO BE 2000 P.S.I. COMPRESSIVE STRENGTH AT 28 DAYS.
9. "T" MANHOLE TO BE MANUFACTURED BY PIPE SUPPLIER.

NO.	REVISIONS	BY	CHK'D	DATE
5				
4				
3				
2				
1				

CITY OF GALT

STANDARD PRECAST "T" MANHOLE

SCALE: 3/8" = 1'-0"	STD. 1-2
DATE: JAN. 10, 1972	
DRAWN BY: T. J. C.	P.ENG.
DESIGN BY: P.A.H.	CITY ENGINEER



NOTES

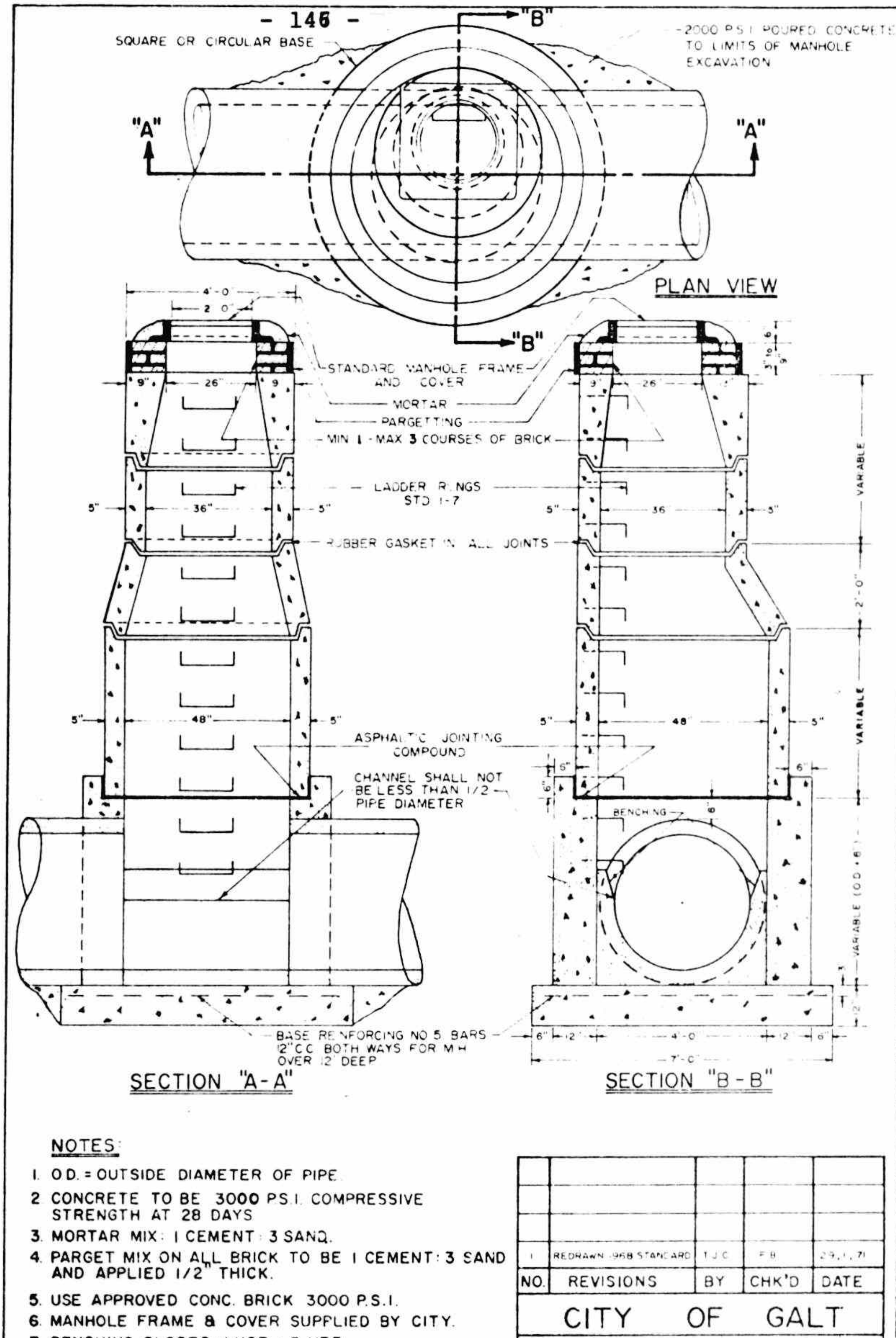
1. O.D = OUTSIDE DIAMETER OF PIPE
2. CONCRETE TO BE 3000 PSI COMPRESSIVE STRENGTH AT 28 DAYS.
3. MORTAR MIX: I CEMENT: 3 SAND.
4. PARGET MIX ON ALL BRICK TO BE I CEMENT: 3 SAND, AND APPLIED 1/2" THICK.
5. USE APPROVED CONC. BRICK 3000 PSI
6. MANHOLE FRAME & COVER SUPPLIED BY CITY.
7. BENCHING SLOPES: I HOR.: 3 VERT.
8. STEPS FIRST STEP 24" MAX. BELOW FRAME.
9. ALL REINFORCING TO BE HI BOND BARS.
10. MIN. COVER OVER REINFORCING OF CONCRETE TO BE 2"
11. ALL BASE SECTIONS TO BE FORMED.

NO.	REVISIONS	BY	CHK'D	DATE
1	REDRAWN 1968 STANDARD	W.B.	F.B.	26.1.71

CITY OF GALT

STANDARD MANHOLE FOR SEWERS OVER 39" I.D.

SCALE: 1/4" = 1'-0"	STD. 1-4
DATE: JAN. 26, 1971	
DRAWN: W. BALL	P.ENG.
DESIGN: F. BANDONI,	CITY ENGINEER



NOTES:

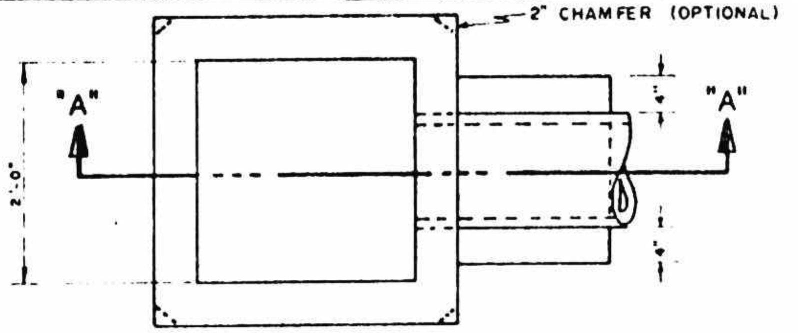
1. O.D. = OUTSIDE DIAMETER OF PIPE.
2. CONCRETE TO BE 3000 P.S.I. COMPRESSIVE STRENGTH AT 28 DAYS
3. MORTAR MIX: 1 CEMENT: 3 SAND.
4. PARGET MIX ON ALL BRICK TO BE 1 CEMENT: 3 SAND AND APPLIED 1/2" THICK.
5. USE APPROVED CONC. BRICK 3000 P.S.I.
6. MANHOLE FRAME & COVER SUPPLIED BY CITY.
7. BENCHING SLOPES: 1 HOR. : 3 VER.
8. STEPS: FIRST STEP 24" MAX. BELOW FRAME.
9. ALL REINFORCING TO BE HI-BOND BARS.
10. MIN. COVER OF 2" OVER REINFORCING OF CONC.
11. ALL BASE SECTIONS TO BE FORMED
12. WALL THICKNESS = 5"

NO.	REVISIONS	BY	CHK'D	DATE
1	REDRAWN 1968 STANDARD	T.J.C.	F.B.	29.1.71

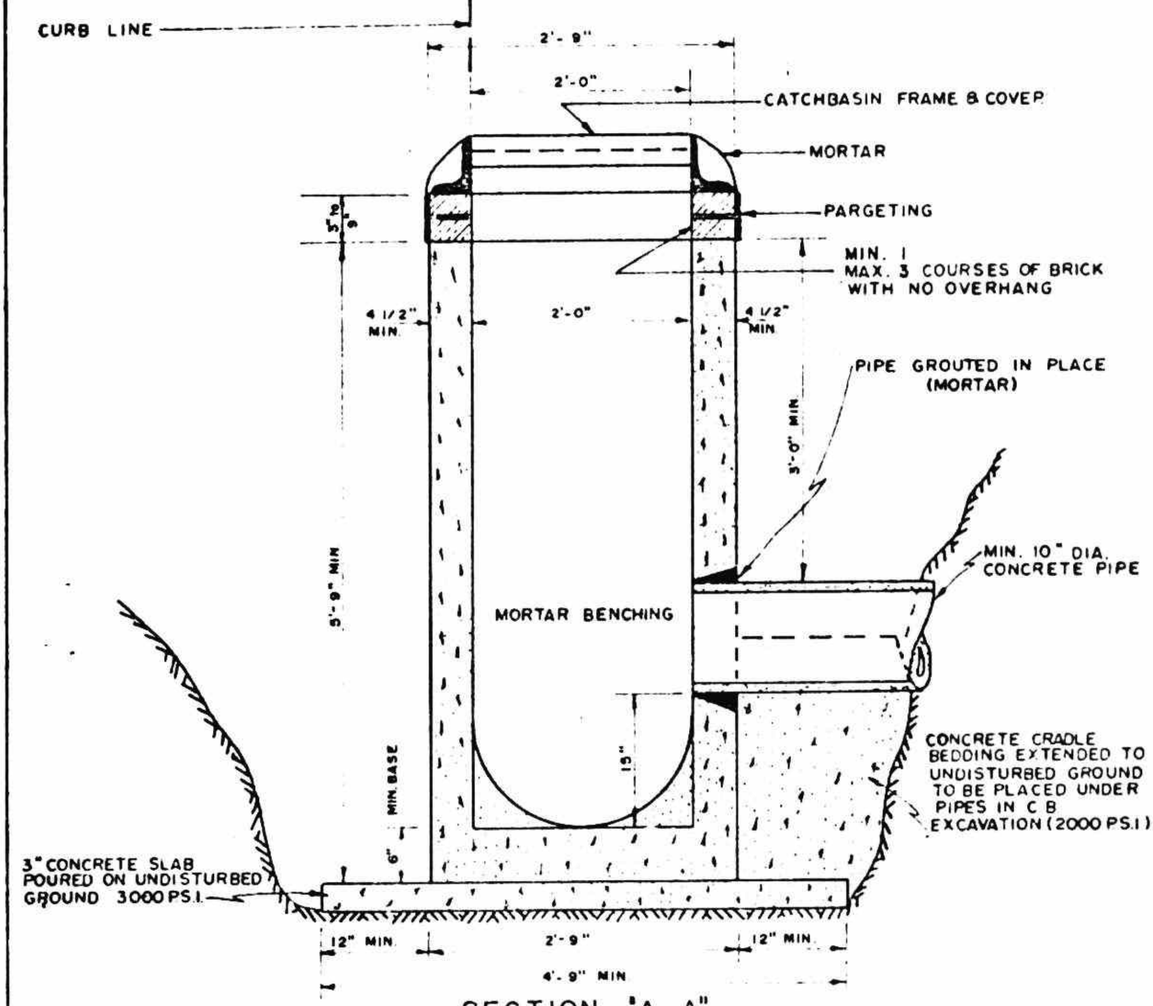
CITY OF GALT

STANDARD MANHOLE FOR SEWERS 39" AND SMALLER AND OVER 8'-0" DEEP

SCALE: 3/8" = 1'-0"	STD. 1-5
DATE: JAN. 29, 1971	
DRAWN: T.J.C.	P.ENG.
DESIGN: F. BANDONI	CITY ENGINEER



PLAN VIEW



SECTION "A-A"

NOTES:

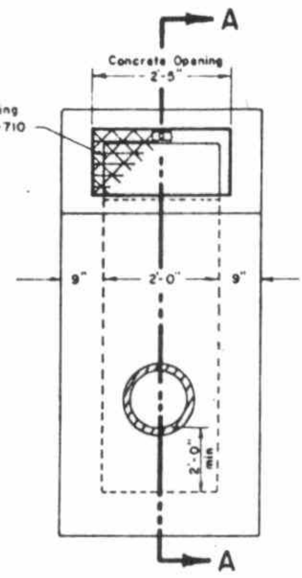
1. CONCRETE TO BE 4000 P.S.I. IN BASIN, WHILE ALL OTHER CONCRETE TO BE 3000 P.S.I. COMPRESSIVE STRENGTH AT 28 DAYS.
2. MORTAR MIX: 1 CEMENT: 3 SAND.
3. PARGET MIX ON ALL BRICK WORK TO BE 1 CEMENT: 3 SAND and APPLIED 1/2" THICK.
4. USE APPROVED CONCRETE BRICK - 3000 P.S.I.
5. CATCHBASIN FRAME and COVER SUPPLIED BY CITY.
6. 4"x4" - 6/6 GAUGE WIRE MESH REINFORCED WALLS and BASE.
7. LIFT HOLES TO BE PLACED IN SIDES OF BASIN.

NO	REVISIONS	BY	CHK'D.	DATE

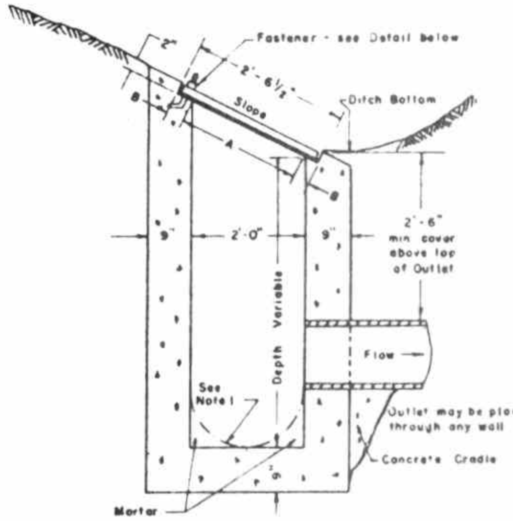
CITY OF GALT

STANDARD PRECAST CATCHBASIN

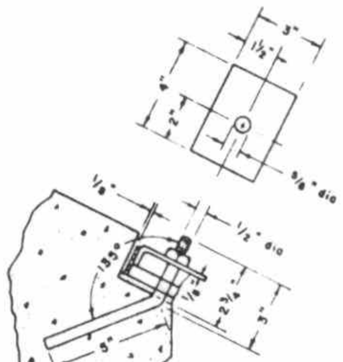
SCALE: 3/4" = 1'-0"	STD. 2-2
DATE: JULY 4, 1968	
DRAWN BY: W. BALL	P. ENG.
DESIGN BY: P. MORROW	CITY ENGINEER



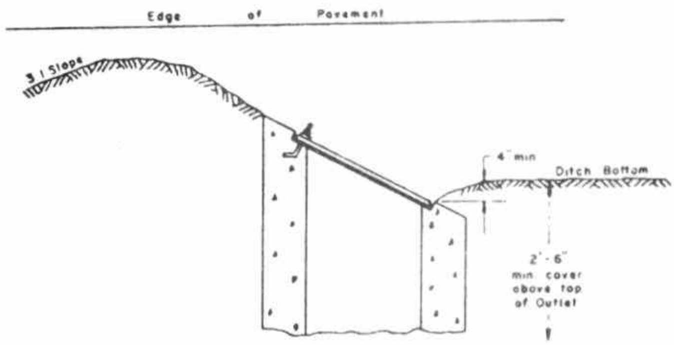
FRONT VIEW



SECTION "A-A"
(Inlet on Ditch Slope)



ENLARGED DETAIL OF GRATING FASTENER
(Steel Plate, Bolt and Nut)



SECTION "A-A"
(Inlet across Ditch)

Slope of Grating	Dimension 'A'	Dimension 'B'
2:1	2'-3"	1¾"
3:1	2'-1½"	2½"
4:1	2'-0¾"	2⅞"

NOTES:

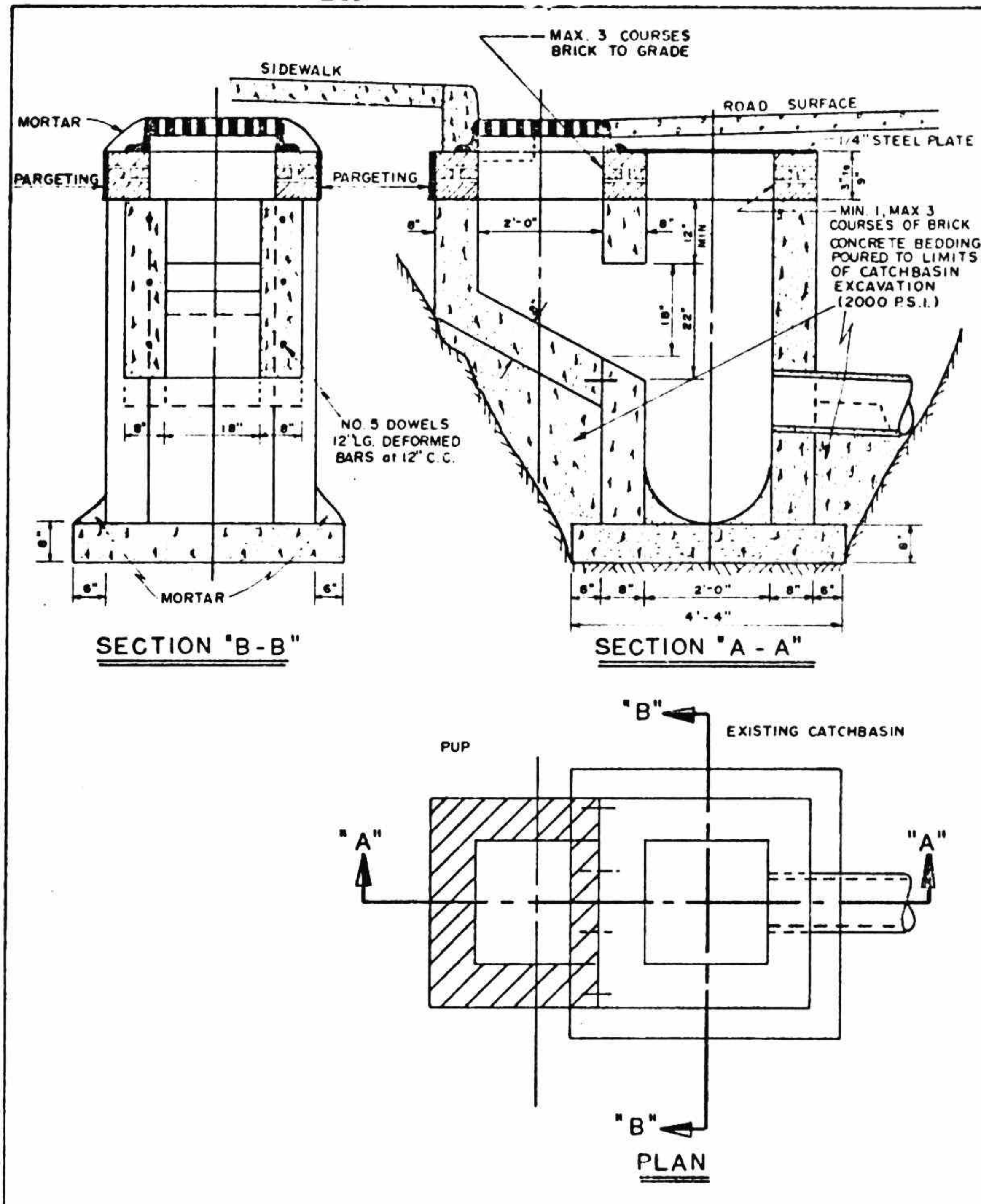
1. TO PERMIT USE OF COLLAPSIBLE FORMS, SEMI-CIRCULAR BOTTOM MAY BE EMPLOYED AT CONTRACTOR'S OPTION.
2. ALL CONCRETE WORK TO CONFORM TO SECTION 9-04 OF D.T.C. FORM 9. CLASS OF CONCRETE: 3000 P.S.I.
3. POROUS BACKFILL TO BE PLACED TO A MINIMUM THICKNESS OF 1 ft. ON ALL SIDES.
4. THIS STANDARD TO BE READ IN CONJUNCTION WITH D.T.C. FORM 407.
5. WHERE 4'X4' INLET BASIN IS REQUIRED, REFER TO D.T.C. STD. DWG. DD-707-C.
6. CONCRETE CRADLE BEDDING UNDER OUTLET PIPE TO BE EXTENDED TO UNDISTURBED GROUND IN CATCHBASIN EXCAVATION AT 3000 P.S.I.
7. MORTAR MIX TO BE: 1 CEMENT : 3 SAND.

NO.	REVISIONS	BY	CHK'D	DATE
5				
4				
3				
2				
1				

CITY OF GALT

STANDARD 2'X2' DITCH INLET CATCH BASIN

SCALE: N. T. S.	STD. 2-3
DATE: JAN. 11, 1972	
DRAWN BY: T. J. C.	[illegible] P.ENG.
DESIGN BY: P. A. H.	CITY ENGINEER



NOTES:

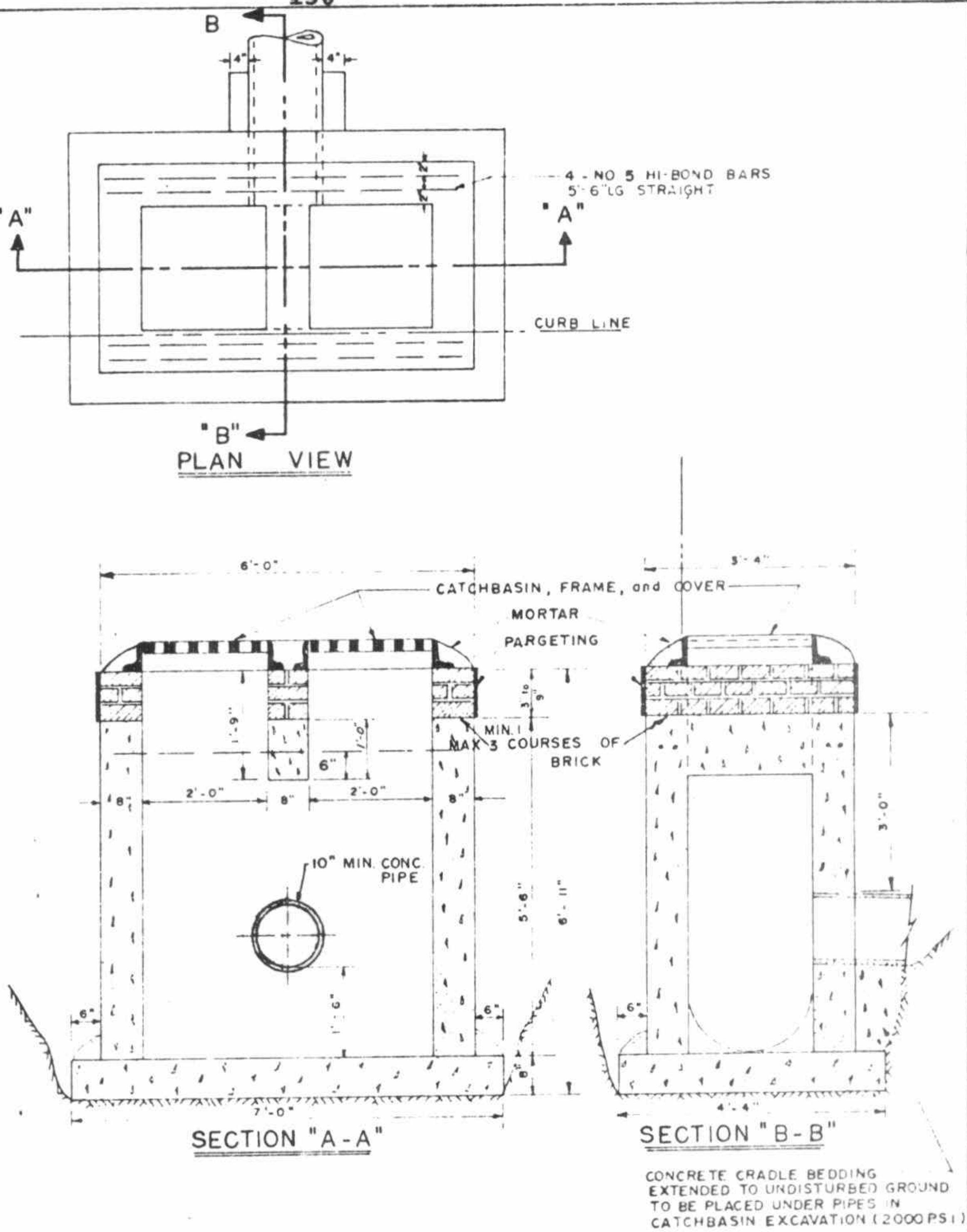
1. CONCRETE TO BE 3000 P.S.I. COMPRESSIVE STRENGTH AT 28 DAYS.
2. MORTAR MIX: 1 CEMENT : 3 SAND.
3. PARGET MIX ON ALL BRICK TO BE 1 CEMENT : 3 SAND & APPLIED 1/2" THICK.
4. USE APPROVED CONCRETE BRICK 3000 P.S.I.
5. CATCHBASIN FRAME & COVER SUPPLIED BY THE CITY.

NO	REVISIONS	BY	CHK'D	DATE

CITY OF GALT

STANDARD (TYPE I)
CATCHBASIN PUP

SCALE: 1/2" = 1'-0"	STD. 2-4
DATE: JULY. 8, 1968	
DRAWN BY: W. BALL	P. ENG.
DESIGN BY: P. MORROW	CITY ENGINEER



NOTES

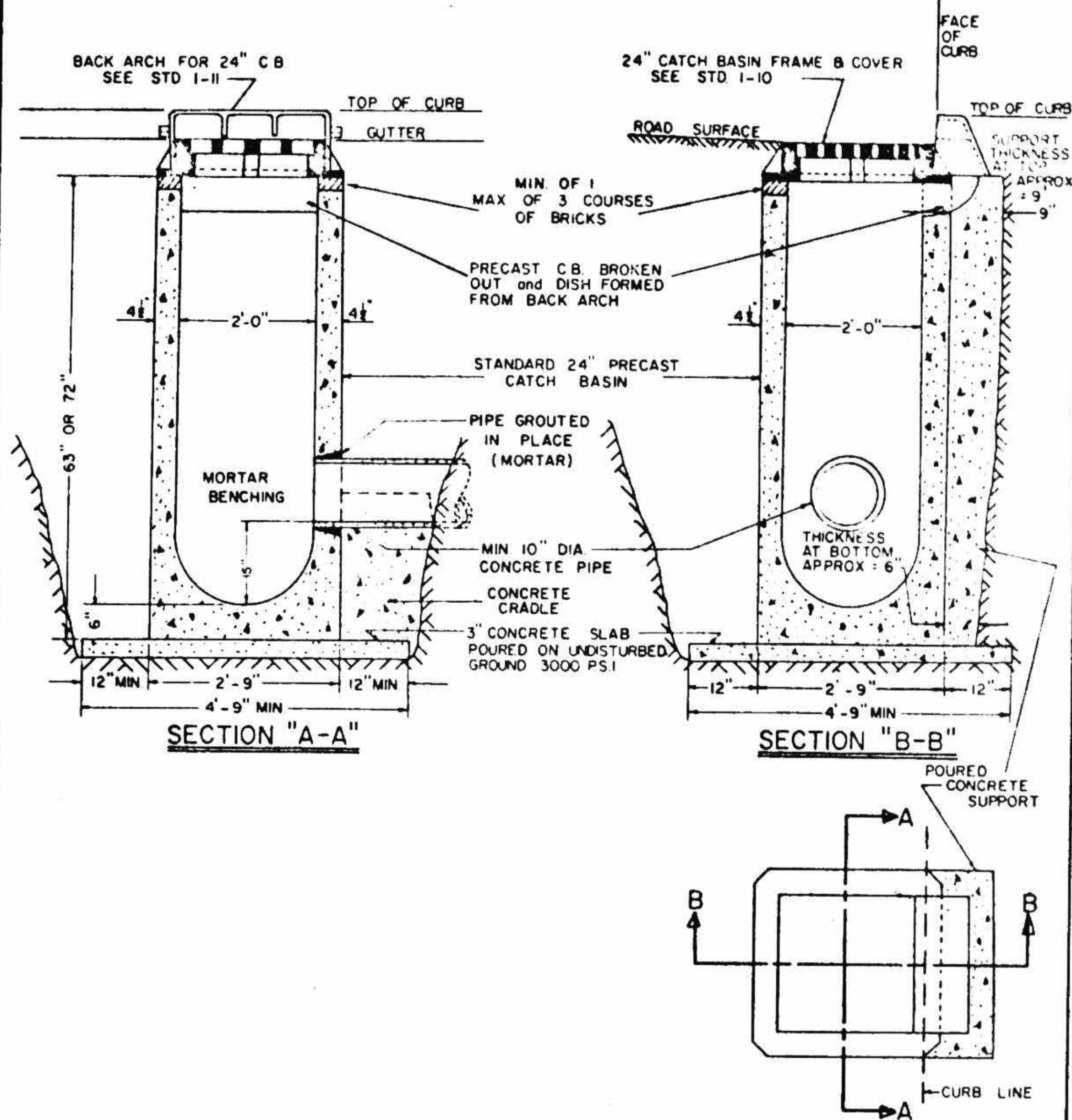
1. CONCRETE TO BE 3000 P.S.I. COMPRESSIVE STRENGTH AT 28 DAYS.
2. MORTAR MIX: 1 CEMENT: 3 SAND
3. PARGET MIX ON ALL BRICK WORK TO BE 1 CEMENT 3 SAND and APPLIED 1/2" THICK.
4. USE APPROVED CONCRETE BRICK 3000 P.S.I.
5. CATCHBASIN FRAME & COVER SUPPLIED BY THE CITY.
6. ALL BASE SECTIONS TO BE FORMED.
7. THIS UNIT MAY ALSO BE PRECAST WITH 4.5" WALLS.

NO	REVISIONS	BY	CHK'D.	DATE

CITY OF GALT

TWIN INLET CATCH BASIN

SCALE: 1/2" = 1'-0"	STD. 2-6(1)
DATE: DEC. 23, 1968	
DRAWN BY: W. BALL	P. ENG.
DESIGN BY: P. J. M.	CITY ENGINEER



NOTES

1. CONCRETE TO BE 4000 P.S.I. IN BASIN, WHILE ALL OTHER CONCRETE TO BE 3000 P.S.I. COMPRESSIVE STRENGTH IN 28 DAYS.
2. MORTAR MIX: 1 CEMENT : 3 SAND.
3. PARGET MIX TO BE 1 CEMENT : 3 SAND and APPLIED ½" THICK TO ALL BRICK WORK.
4. USE APPROVED CONCRETE BRICK - 3000 P.S.I.
5. CATCH BASIN FRAME and COVER and BACK ARCH SUPPLIED BY CITY.
6. RE-INFORCING WIRE - A.S.T.M. no. A185-58T W.W.M. 4" x 4" - 6/6.
7. LIFT HOLES TO BE PLACED IN SIDES OF BASIN.
8. CONCRETE CRADLE BEDDING EXTENDING TO UNDISTURBED GROUND TO BE PLACED UNDER PIPES IN C.B. EXCAVATION (2000 P.S.I.).

NO.	REVISIONS	BY	CHK'D	DATE

CITY OF GALT

24" PRECAST CATCH BASIN WITH BACK ARCH

SCALE: 1/2" = 1'	STD. 2-8
DATE: FEB 9, 1970	
DRAWN BY: T.J.C.	P. ENG.
DESIGN BY: F. BANDONI	CITY ENGINEER

NOTE

1. CONCRETE TO BE 3000 P.S.I.
2. CATCH BASIN FRAME AND COVER TO BE OF TYPE STD 1-10.
3. CATCH BASIN TO BE OF TYPE STD 2-8.
4. ANGLE FOR C.B. TO BE SET IN ACCORDANCE WITH THE FOLLOWING ROAD GRADES:

5-6 %	60°
6-7 %	70°
7—	80°

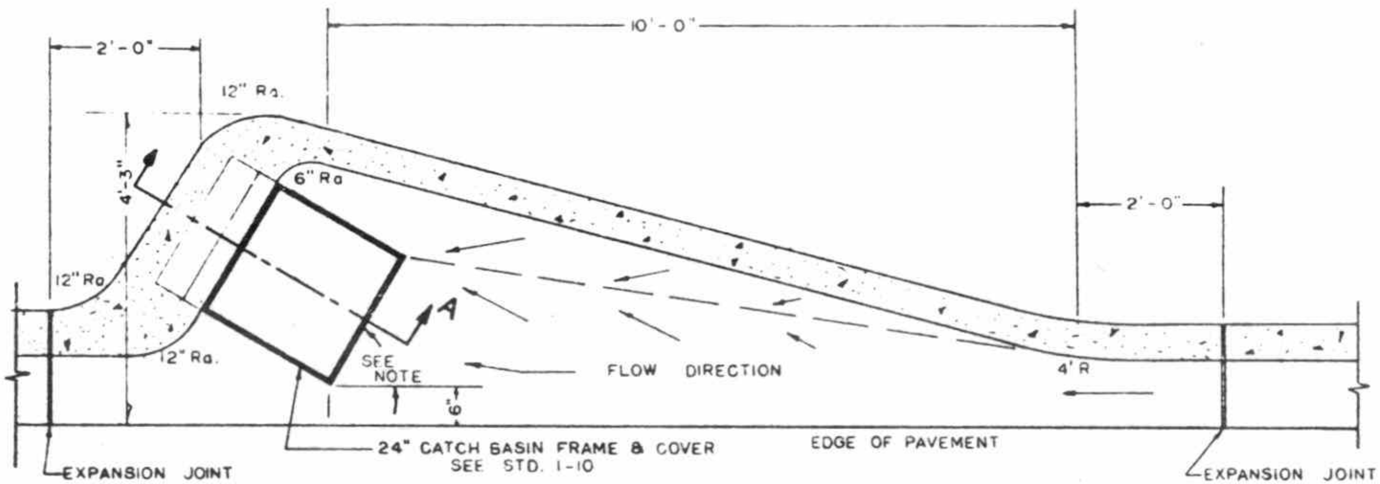
5. C.B. OPEN BACK ARCH TO BE OF TYPE STD 1-11.

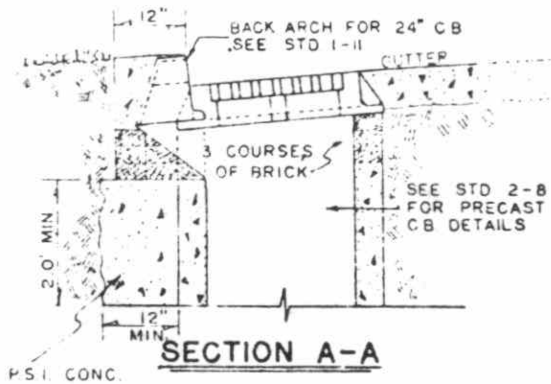
1	CURB RD. B. ERICKS	Ed S	P.H.	20.6.72
NO	REVISIONS	BY	CHK'D	DATE

CITY OF GALT

SET-BACK CATCH BASIN

SCALE 1/2" = 1'	STD. 2-9
DATE: APR 5, 1970	
DRAWN BY T.J.C.	P. ENG
DESIGN D. SANDIESON	CITY ENGINEER





SECTION A-A

FIG I

REINFORCED CONCRETE ENCASEMENT

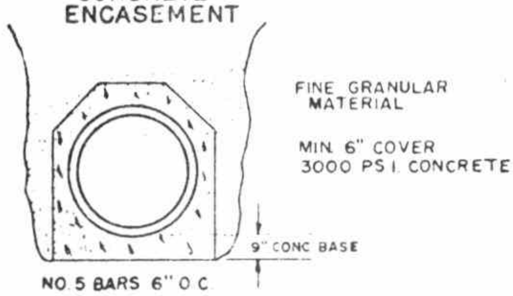


FIG. 4

CLASS B BEDDING 3/4" GRANULAR MATERIAL

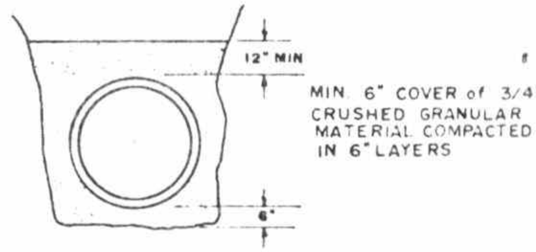


FIG. 2

CLASS AA BEDDING FULL CONCRETE ENCASEMENT

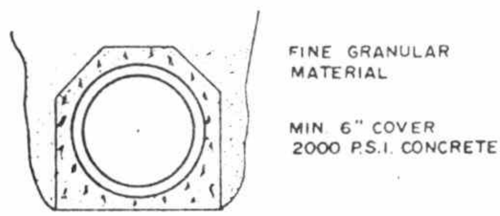


FIG. 5

CLASS AB BEDDING CONCRETE TOP ENCASEMENT

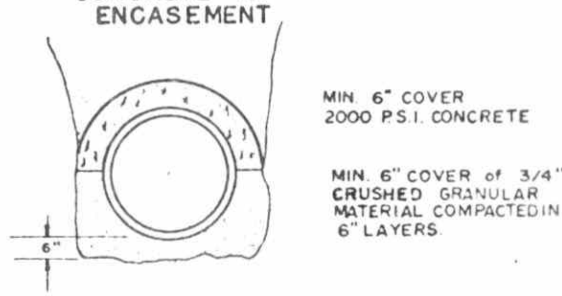


FIG. 3

CLASS A BEDDING CONCRETE CRADLE

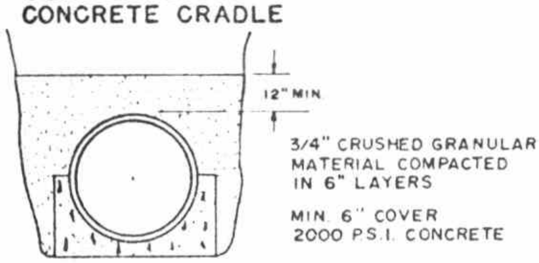
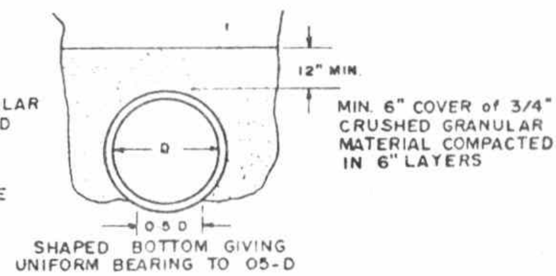


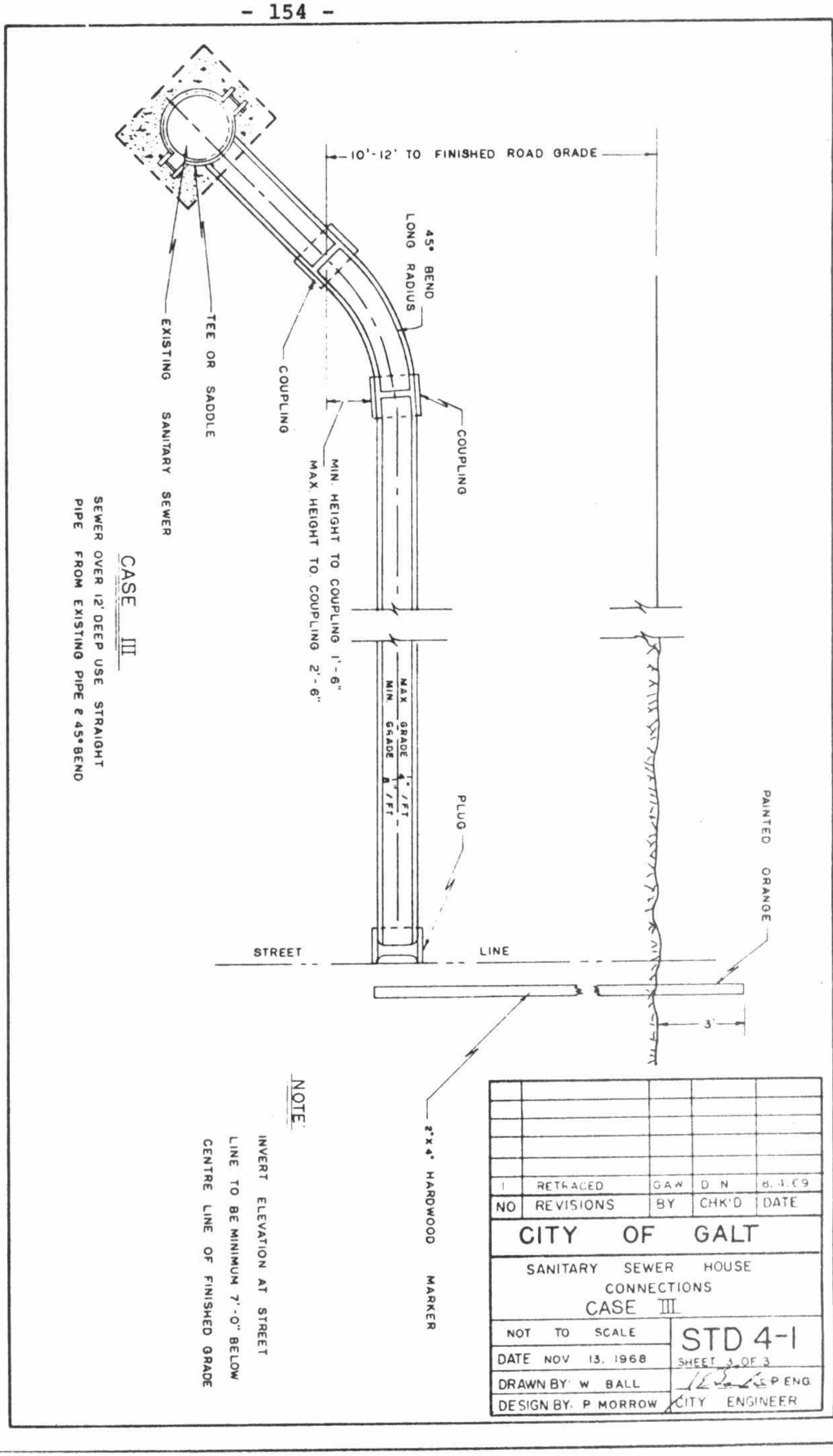
FIG. 6

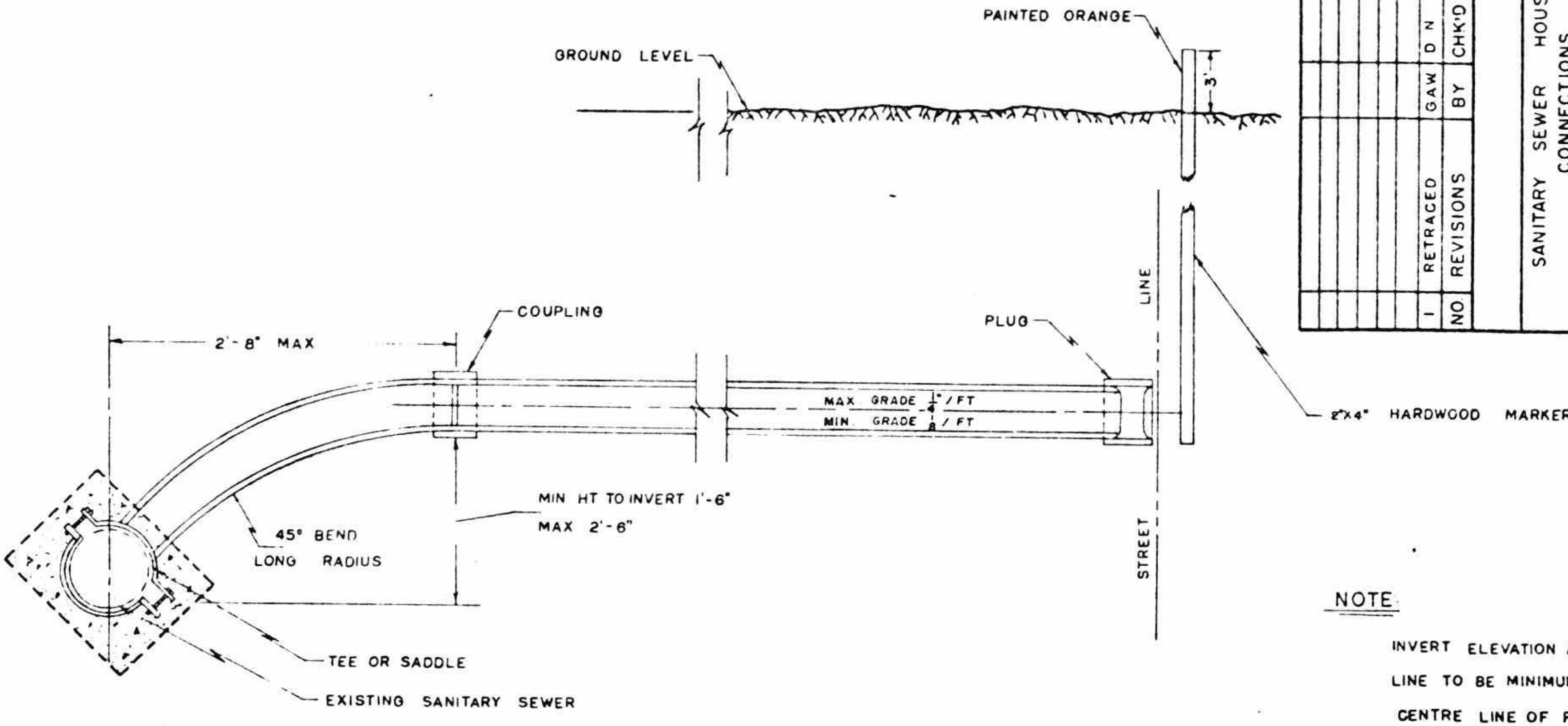
CLASS C BEDDING SHAPED TRENCH BOTTOM



NO	REVISIONS	BY	CHK. D.	DATE

CITY OF GALT	
STANDARD BEDDING for PIPE LAYING	
NOT TO SCALE	STD. 3-1
DATE: OCT. 16, 1968	
DRAWN BY: W. BALL	[signature] P. ENG.
DESIGN BY: D. NARAINE	CITY ENGINEER


10'-12' TO FINISHED ROAD GRADE
45° BEND LONG RADIUS
COUPLING
COUPLING
TEE OR SADDLE
EXISTING SANITARY SEWER
MIN. HEIGHT TO COUPLING 1'-6"
MAX. HEIGHT TO COUPLING 2'-6"
MAX GRADE 1" /FT
MIN. GRADE ⅛" /FT
PLUG
STREET LINE
PAINTED ORANGE
3'
2"x4" HARDWOOD MARKER
CASE III
SEWER OVER 12' DEEP USE STRAIGHT PIPE FROM EXISTING PIPE & 45° BEND
NOTE:
INVERT ELEVATION AT STREET LINE TO BE MINIMUM 7'-0" BELOW CENTRE LINE OF FINISHED GRADE
1 RETRACED GAW D N 8.4.69
NO REVISIONS BY CHK'D DATE
CITY OF GALT
SANITARY SEWER HOUSE CONNECTIONS
CASE III
NOT TO SCALE
STD 4-1
DATE NOV 13, 1968
SHEET 3 OF 3
DRAWN BY W BALL
P ENG
DESIGN BY P MORROW
CITY ENGINEER



CASE II

CONNECTION TO SEWER 10' - 12' DEEP

USE 45° BEND

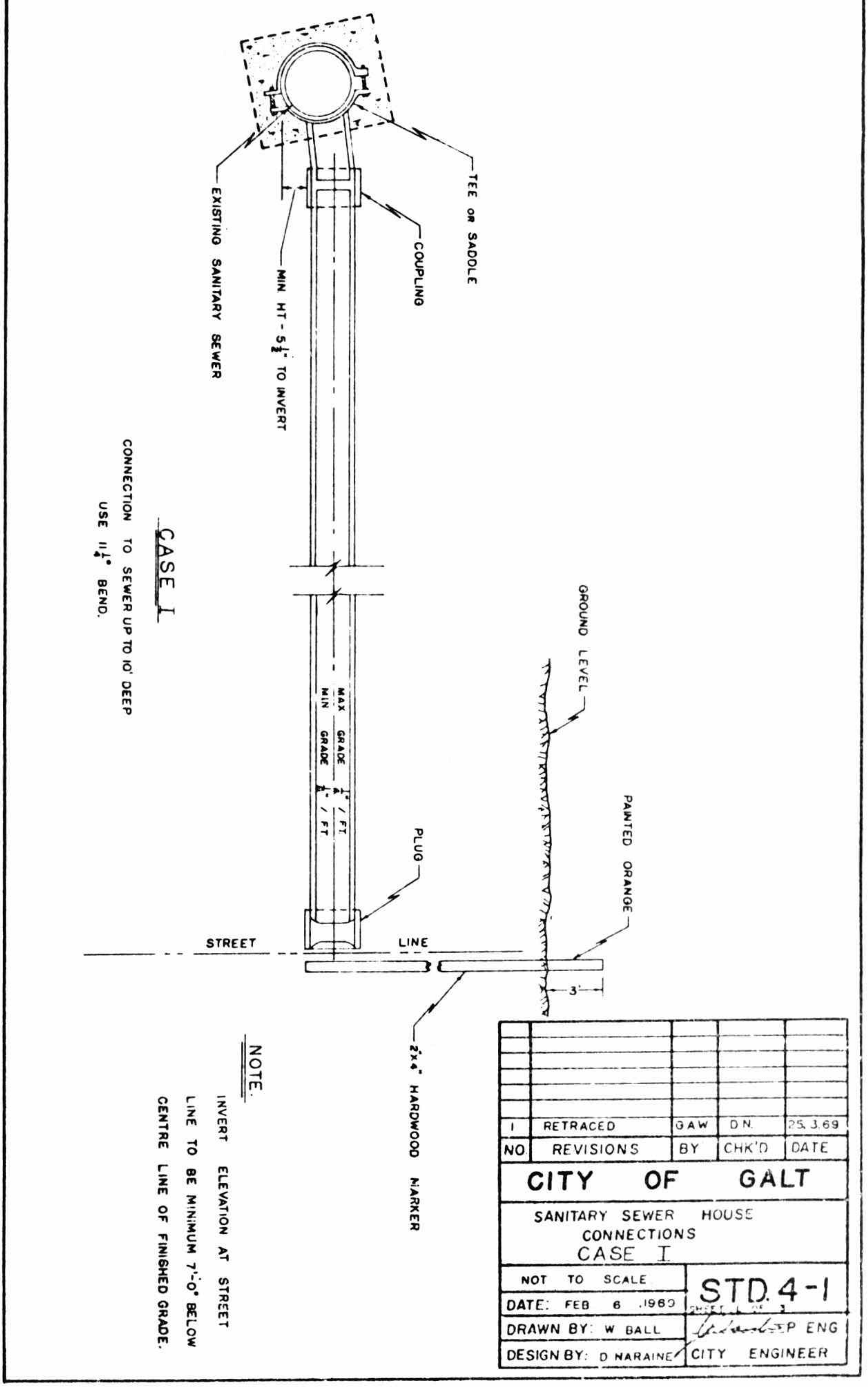
NOTE

INVERT ELEVATION AT STREET LINE TO BE MINIMUM 7'-0" BELOW CENTRE LINE OF FINISHED GRADE.

NO	REVISIONS	BY	CHK'D	DATE
1	RETRACED	GAW	D N	2.4.69

SANITARY SEWER HOUSE CONNECTIONS
CASE II

NOT TO SCALE	STD - 4-1
DATE: APRIL 2, 1969	SHEET 2 OF 3
DRAWN BY: W BALL	[signature] ENG
DESIGN BY: D NARAINE	CITY ENGINEER


TEE OR SADDLE
COUPLING
EXISTING SANITARY SEWER
MIN HT - 5½" TO INVERT
CASE I
CONNECTION TO SEWER UP TO 10' DEEP
USE 11¼° BEND.
GROUND LEVEL
MAX GRADE
MIN GRADE
PLUG
PAINTED ORANGE
STREET LINE
3'
2"x4" HARDWOOD MARKER
NOTE.
INVERT ELEVATION AT STREET
LINE TO BE MINIMUM 7'-0" BELOW
CENTRE LINE OF FINISHED GRADE.
1 RETRACED GAW D.N. 25.3.69
NO. REVISIONS BY CHK'D DATE
CITY OF GALT
SANITARY SEWER HOUSE
CONNECTIONS
CASE I
NOT TO SCALE
STD. 4-1
DATE: FEB 6 1969
DRAWN BY: W BALL
DESIGN BY: D NARAINE
CITY ENGINEER

HYDRAULICS AND DESIGN OF SEWERS

by

Mr. J. L. Tersigni, P. Eng.

Hydraulics and Design of Sewers

This lecture is part of a design course on sewers and water mains and will follow a lecture on "Predicting Flows for Storm and Sanitary Sewers and the Physical Features of Sewer Systems".

The highlights of Chapters 5 and 6 of the W.P.C.F. Manual of Practice No. 9, "Hydraulics of Sewers" and "Design of Sewer Systems" are covered here for easy reference. For a more detailed look into the subject it is suggested that the W.P.C.F. Manual be studied.

Some of the topics that are dealt with in this part of the course are the Mannings formula and values of 'n', minimum and maximum design velocities, hydraulics of culverts, hydraulics of junction and transition manholes, and hydrogen sulphide generation. An illustration is given to show how sewer sizes, grades and depths are determined.

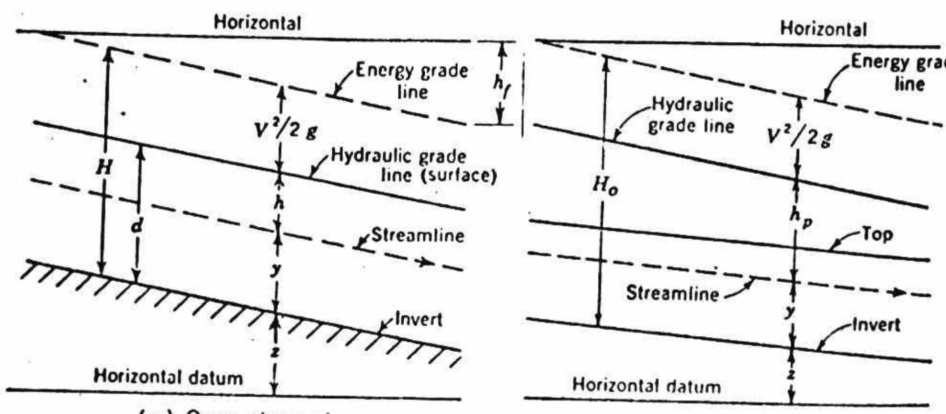
Hydraulic Considerations

Hydraulically sewers are either classed as open-channels or pressure conduits. The liquid surface in open channels is exposed to the atmosphere. In pressure conduits the liquid fully fills its conduit. Sewers are most often designed as open channels. It may sometimes be advantageous to design storm and sanitary sewers to operate under a head or as pressure conduits. We will concentrate on open-channel flow today.

(i) Type of Flow

We have steady flow and unsteady flow. Steady flow may be uniform if the velocity and depth are constant from point to point along an open channel or if the cross sectional area is constant in pressure conduits. See Figure 17 on Page 3 for steady uniform flow types.

Other types of flow we experience but will not study in this lecture are laminar, turbulent, subcritical or supercritical.



(a) Open channel (b) Closed conduit flow

FIGURE 17.—Comparison of uniform flow in an open-channel and a pressure conduit.

(ii) Specific Energy Head

From the energy principle the relationship can be developed that $H_o = d + \frac{V^2}{2g}$ for open channel flow, where H_o is the specific energy head, d is the depth of flow, V the mean velocity and g the acceleration due to gravity. The term $\frac{V^2}{2g}$ is commonly referred to as the velocity head.

(iii) Classification of Flow Profiles

Critical depth serves as a useful tool for the classification of all open channels. When the rate of flow is maximum for a given total energy the condition is referred to as critical flow or flow at critical depth. This point is most useful for calculation of drawdown or backwater curves. Flow at velocities greater than critical is called supercritical flow and flow at velocities less than critical is called subcritical flow.

The normal depth of flow in an open channel is that depth at which the slope of the water surface and the slope of bottom are parallel.

Hydraulic Formulas

The Kutter and Manning formulas have been widely used in sewage-flow computations for open channel conditions. The Manning, Hazen-Williams and Darcy-Weisbach formulas are more commonly used for pressure conduits.

Charts have been prepared for the Kutter and Manning formulas and are included on Pages 6 to 13 inclusive for easy reference. It is recommended that Manning's formula be used for sewer design because of its simplicity and the close approximation of the Manning 'n' and the Kutter 'n'. For example the Kutter formula gives a Q of 31.33 c.f.s. and a V of 7.88 f.p.s. for a 30 inch diameter pipe @ 1.00% grade, 'n' = 0.013 (see Chart on Page 6) while the Manning formula gives values for Q of 31.0 c.f.s. and for V of 7.78 f.p.s. (see Chart on Page 8). It should be noted that the Chart on Page 6 has been developed using 'n' = 0.015 for pipe sizes up to and including 24 inches in diameter and 'n' = 0.013 for pipes of larger diameter.

KUTTER'S FORMULA

$Q = A c \sqrt{R \times S}$ $\quad V = C \sqrt{R \times S}$

DISCHARGE AND VELOCITY FOR CIRCULAR PIPES

n = 0.015

GRADE %	8" Q	8" V	10" Q	10" V	12" Q	12" V	15" Q	15" V	18" Q	18" V	21" Q	21" V
6.00	2.23	6.38	4.17	7.64	6.94	8.83	12.86	10.48	21.26	12.03	32.49	13.51
5.90	2.21	6.34	4.13	7.58	6.88	8.77	12.74	10.38	20.90	11.83	32.05	13.33
5.80	2.19	6.29	4.09	7.51	6.82	8.69	12.62	10.28	20.73	11.73	31.78	13.22
5.70	2.17	6.23	4.06	7.44	6.77	8.63	12.53	10.21	20.54	11.63	31.50	13.10
5.60	2.16	6.18	4.02	7.38	6.71	8.55	12.41	10.12	20.36	11.52	31.23	12.98
5.50	2.14	6.12	3.99	7.31	6.64	8.46	12.31	10.03	20.15	11.40	30.95	12.87
5.40	2.12	6.07	3.95	7.25	6.52	8.38	12.19	9.93	20.00	11.32	30.67	12.75
5.30	2.10	6.01	3.91	7.18	6.52	8.31	12.07	9.83	19.80	11.21	30.37	12.63
5.20	2.08	5.95	3.87	7.11	6.46	8.23	11.95	9.74	19.62	11.11	30.09	12.51
5.10	2.06	5.89	3.84	7.04	6.40	8.15	11.84	9.65	19.43	11.00	29.80	12.39
5.00	2.04	5.83	3.80	6.96	6.34	8.07	11.66	9.50	19.28	10.87	29.66	12.30
4.90	2.02	5.78	3.76	6.90	6.27	7.99	11.60	9.45	19.04	10.78	29.21	12.15
4.80	2.00	5.72	3.72	6.83	6.22	7.92	11.48	9.34	18.94	10.72	28.91	12.02
4.70	1.97	5.66	3.68	6.76	6.15	7.83	11.38	9.28	18.67	10.57	28.62	11.90
4.60	1.95	5.60	3.64	6.69	6.07	7.73	11.25	9.17	18.45	10.44	28.30	11.77
4.50	1.93	5.54	3.60	6.61	6.01	7.66	11.11	9.05	18.25	10.33	27.99	11.64
4.40	1.91	5.47	3.56	6.54	5.97	7.57	11.01	8.97	18.01	10.19	27.67	11.51
4.30	1.89	5.41	3.52	6.46	5.86	7.47	10.87	8.86	17.84	10.10	27.38	11.38
4.20	1.87	5.35	3.48	6.39	5.81	7.40	10.74	8.76	17.63	9.98	27.08	11.26
4.10	1.84	5.28	3.44	6.31	5.73	7.30	10.62	8.66	17.42	9.86	26.72	11.11
4.00	1.82	5.20	3.40	6.24	5.66	7.21	10.42	8.49	17.30	9.78	26.53	11.03
3.90	1.80	5.15	3.36	6.16	5.59	7.12	10.32	8.41	16.99	9.61	26.06	10.84
3.80	1.78	5.09	3.31	6.08	5.52	7.03	10.24	8.34	16.78	9.49	25.72	10.69
3.70	1.75	5.02	3.27	6.00	5.45	6.94	10.05	8.19	16.55	9.37	25.38	10.55
3.60	1.73	4.95	3.22	5.91	5.37	6.84	9.95	8.11	16.33	9.24	25.04	10.41
3.50	1.70	4.88	3.18	5.83	5.30	6.75	9.77	7.98	16.10	9.11	24.68	10.26
3.40	1.68	4.81	3.13	5.75	5.22	6.65	9.66	7.88	15.86	8.98	24.34	10.12
3.30	1.65	4.74	3.09	5.66	5.14	6.55	9.52	7.76	15.62	8.84	23.96	9.96
3.20	1.63	4.75	3.04	5.58	5.06	6.45	9.38	7.65	15.40	8.71	23.60	9.81
3.10	1.60	4.59	2.99	5.49	4.99	6.35	9.23	7.52	15.14	8.57	23.22	9.66
3.00	1.57	4.49	2.92	5.36	4.90	6.24	9.08	7.35	15.02	8.50	22.92	9.53
2.90	1.55	4.44	2.89	5.31	4.82	6.14	8.93	7.27	14.68	8.31	22.47	9.34
2.80	1.52	4.37	2.84	5.22	4.74	6.04	8.78	7.15	14.40	8.15	22.08	9.18
2.70	1.50	4.29	2.79	5.12	4.65	5.93	8.62	7.02	14.13	8.00	21.68	9.01
2.60	1.47	4.21	2.74	5.03	4.56	5.82	8.46	6.89	13.87	7.85	21.27	8.84
2.50	1.44	4.12	2.69	4.93	4.48	5.70	8.29	6.76	13.60	7.70	20.85	8.67
2.40	1.41	4.04	2.63	4.83	4.39	5.59	8.12	6.62	13.33	7.54	20.43	8.50
2.30	1.38	3.96	2.58	4.73	4.29	5.47	7.95	6.48	13.04	7.38	19.99	8.31
2.20	1.35	3.87	2.52	4.62	4.20	5.35	7.78	6.34	12.76	7.22	19.56	8.13
2.10	1.32	3.78	2.46	4.52	4.10	5.22	7.60	6.19	12.46	7.05	19.12	7.95
2.00	1.28	3.66	2.38	4.37	3.99	5.08	7.34	5.98	12.25	6.93	18.69	7.77
1.90	1.25	3.59	2.34	4.30	3.90	4.97	7.23	5.89	11.85	6.71	18.19	7.56
1.80	1.22	3.50	2.28	4.18	3.80	4.84	7.03	5.73	11.54	6.53	17.69	7.36
1.70	1.18	3.37	2.19	4.03	3.68	4.69	6.76	5.51	11.27	6.30	17.22	7.16
1.60	1.15	3.30	2.15	3.94	3.58	4.56	6.63	5.40	10.88	6.16	16.68	6.94
1.50	1.11	3.19	2.08	3.82	3.47	4.41	6.42	5.23	10.53	5.96	16.26	6.76
1.43	1.09	3.10	2.03	3.71	3.38	4.30	6.26	5.08	10.22	5.80	15.76	6.57
1.40	1.08	3.08	2.01	3.69	3.35	4.26	6.20	5.05	10.17	5.76	15.61	6.49
1.30	1.04	2.97	1.93	3.55	3.23	4.11	5.97	4.87	9.80	5.55	15.04	6.25
1.25	1.02	2.91	1.89	3.48	3.17	4.03	5.86	4.76	9.61	5.44	14.74	6.14
1.20	1.00	2.86	1.86	3.42	3.10	3.95	5.74	4.68	9.42	5.33	14.44	6.00
1.11	0.95	2.74	1.79	3.27	2.99	3.81	5.53	4.52	9.05	5.12	13.90	5.79
1.10	0.95	2.74	1.78	3.26	2.97	3.78	5.49	4.48	9.01	5.10	13.83	5.75
1.00	0.91	2.59	1.69	3.10	2.83	3.60	5.20	4.24	8.68	4.91	13.25	5.51
0.98	0.90	2.58	1.68	3.08	2.80	3.57	5.18	4.22	8.51	4.81	13.05	5.43
0.95	0.88	2.54	1.65	3.04	2.85	3.51	5.10	4.15	8.37	4.73	12.84	5.33
0.92	0.87	2.50	1.63	2.99	2.71	3.45	5.02	4.09	8.24	4.66	12.63	5.25
0.90	0.85	2.43	1.58	2.90	2.64	3.36	4.90	3.99	8.13	4.60	12.40	5.16
0.87	0.83	2.38	1.55	2.84	2.58	3.29	4.80	3.91	7.96	4.50	12.20	5.05
0.83	0.81	2.33	1.52	2.78	2.53	3.22	4.69	3.83	7.78	4.41	11.90	4.94
0.80	0.79	2.28	1.48	2.72	2.47	3.15	4.59	3.74	7.61	4.31	11.60	4.84
0.77	0.78	2.25	1.47	2.69	2.45	3.12	4.54	3.70	7.53	4.26	11.50	4.78
0.74	0.77	2.20	1.43	2.63	2.39	3.04	4.44	3.62	7.36	4.16	11.20	4.67
0.71	0.76	2.17	1.42	2.60	2.36	3.01	4.38	3.57	7.27	4.11	11.10	4.62
0.69	0.74	2.12	1.38	2.54	2.31	2.94	4.28	3.49	7.10	4.02	10.90	4.51
0.67	0.73	2.10	1.37	2.51	2.28	2.90	4.23	3.45	7.01	3.97	10.70	4.46
0.65	0.71	2.04	1.33	2.44	2.22	2.83	4.13	3.36	6.84	3.87	10.50	4.35
0.63	0.705	2.02	1.32	2.41	2.20	2.80	4.07	3.32	6.76	3.82	10.30	4.29
0.61	0.70	2.00	1.30	2.38	2.17	2.76	4.02	3.28	6.67	3.78	10.20	4.24
0.59	0.69	1.97	1.28	2.35	2.14	2.73	3.97	3.24	6.59	3.73	10.10	4.18
0.57	0.68	1.94	1.27	2.32	2.11	2.69	3.92	3.20	6.50	3.68	9.93	4.13
0.56	0.67	1.92	1.25	2.29	2.08	2.66	3.87	3.15	6.42	3.63	9.80	4.08
0.54	0.66	1.89	1.23	2.26	2.06	2.62	3.82	3.11	6.33	3.58	9.67	4.02
0.53	0.65	1.87	1.22	2.23	2.03	2.58	3.77	3.07	6.24	3.53	9.54	3.97
0.51	0.64	1.84	1.20	2.20	2.00	2.55	3.71	3.03	6.16	3.49	9.41	3.91
0.50	0.63	1.82	1.18	2.17	1.97	2.51	3.66	2.98	6.07	3.44	9.28	3.86
0.48	0.62	1.77	1.15	2.11	1.92	2.44	3.56	2.90	5.90	3.34	9.02	3.75
0.45	0.60	1.71	1.12	2.05	1.86	2.37	3.46	2.82	5.73	3.24	8.78	3.64
0.43	0.59	1.69	1.10	2.02	1.83	2.34	3.40	2.77	5.65	3.20	8.63	3.59
0.42	0.58	1.66	1.08	1.99	1.81	2.30	3.35	2.73	5.56	3.15	8.49	3.53
0.40	0.56	1.61	1.05	1.92	1.75	2.23	3.25	2.65	5.39	3.05	8.23	3.42
0.38	0.55	1.59	1.03	1.89	1.72	2.19	3.20	2.61	5.30	3.00	8.10	3.37
0.37	0.54	1.56	1.02	1.86	1.70	2.16	3.15	2.56	5.22	2.95	7.97	3.31
0.36	0.536	1.53	0.99	1.83	1.67	2.12	3.09	2.52	5.13	2.90	7.84	3.26
0.34	0.53	1.51	0.98	1.80	1.64	2.09	3.04	2.48	5.05	2.86	7.71	3.21
0.33	0.52	1.48	0.97	1.77	1.61	2.05	2.99	2.44	4.96	2.80	7.58	3.15
0.32			0.95	1.74	1.58	2.02	2.94	2.40	4.88	2.76	7.45	3.10
0.31			0.93	1.71	1.56	1.98	2.89	2.35	4.79	2.71	7.32	3.04
0.30			0.92	1.68	1.53	1.95	2.84	2.31	4.70	2.66	7.19	2.99
0.29			0.90	1.65	1.50	1.91	2.79	2.27	4.62	2.61	7.06	2.93
0.285			0.89	1.63	1.48	1.89	2.75	2.24	4.57	2.59	6.98	2.90
0.28			0.88	1.61	1.46	1.87	2.72	2.22	4.51	2.55	6.89	2.86
0.27			0.87	1.59	1.45	1.84	2.68	2.19	4.45	2.52	6.80	2.83
0.26			0.85	1.57	1.43	1.82	2.65	2.16	4.39	2.48	6.70	2.79
0.255			0.84	1.55	1.41	1.79	2.61	2.13	4.33	2.45	6.61	2.75
0.25			0.83	1.53	1.38	1.77	2.58	2.10	4.28	2.42	6.53	2.72
0.24			0.81	1.49	1.36	1.73	2.52	2.05	4.17	2.36	6.38	2.65
0.23					1.33	1.69	2.46	2.00	4.08	2.31	6.23	2.59
0.22					1.30	1.65	2.40	1.96	3.99	2.26	6.09	2.53
0.21					1.27	1.61	2.35	1.92	3.90	2.21	5.98	2.48
0.20					1.24	1.58	2.30	1.88	3.82	2.16	5.84	2.43
0.167					1.13	1.44	2.10	1.72	3.49	1.98	5.33	2.22
0.143							1.95	1.59	3.23	1.83	4.94	2.09
0.125							1.81	1.48	3.00	1.70	4.59	1.91
0.10							1.63	1.33	2.70	1.53	4.13	1.72

n = 0.013

GRADE %	24" Q	24" V	27" Q	27" V	30" Q	30" V	33" Q	33" V	36" Q	36" V	42" Q	42" V	48" Q	48" V	54" Q	54" V	60" Q	60" V
6.00	46.87	14.92																
5.90	46.49	14.80																
5.80	46.10	14.67																
5.70	45.71	14.55																
5.60	45.30	14.42																
5.50	44.91	14.29																
5.40	44.47	14.15																
5.30	44.07	14.03																
5.20	43.64	13.89																
5.10	43.23	13.76																
5.00	42.82	13.63																
4.90	42.36	13.48																
4.80	41.93	13.34																
4.70	41.49	13.21																
4.60	41.07	13.07																
4.50	40.60	12.92																
4.40	40.14	12.78																
4.30	39.71	12.64																
4.20	39.22	12.48																
4.10	38.27	12.18																
4.00	38.04	12.10																
3.90	37.81	12.03																
3.80	37.38	11.90																
3.70	36.81	11.72																
3.60	36.32	11.56																
3.50	35.81	11.40	58.61	14.74														
3.40	35.29	11.23	57.76	14.53														
3.30	34.78	10.07	56.91	14.31														
3.20	34.24	10.90	56.04	14.10														
3.10	33.69	10.72	55.13	13.87														
3.00	33.11	10.54	54.19	13.63	72.11	14.69	93.13	15.68										
2.90	32.58	10.37	53.35	13.42	71.03	14.47	91.78	15.45										
2.80	31.99	10.18	52.41	13.18	69.80	14.22	90.20	15.19										
2.70	31.42	10.00	51.47	12.95	68.54	13.96	88.58	14.92										
2.60	30.85	9.82	50.51	12.70	67.22	13.69	86.87	14.63										
2.50	30.27	9.63	49.48	12.45	65.94	13.43	85.21	14.35	107.74	15.24								
2.40	29.66	9.44	48.53	12.21	64.62	13.16	83.46	14.05	105.60	14.94								
2.30	29.01	9.23	47.48	11.91	63.25	12.88	81.70	13.76	103.37	14.63								
2.20	28.38	9.03	46.46	11.68	61.88	12.60	80.01	13.47	101.09	14.30								
2.10	27.73	8.83	45.37	11.41	60.45	12.31	78.06	13.14	98.74	13.97								
2.00	26.99	8.59	44.17	11.11	58.76	11.97	75.90	12.78	96.06	13.59	145.28	15.10	207.83	16.54				
1.90	26.37	8.39	43.15	10.85	57.46	11.71	74.26	12.50	93.86	13.28	141.72	14.73	203.26	16.18	278.11	17.49	369.09	18.80
1.80	25.67	8.17	42.02	10.57	55.96	11.40	72.28	12.17	91.42	12.93	137.94	14.34	197.86	15.75	270.11	16.98	359.20	18.29
1.70	24.88	7.92	40.75	10.25	54.14	11.03	70.02	11.79	88.57	12.53	134.02	13.93	191.64	15.25	262.10	16.48	349.50	17.80
1.60	24.19	7.70	39.61	9.97	52.74	10.74	68.12	11.47	86.22	12.20	130.04	13.52	186.50	14.84	255.27	16.05	338.61	17.25
1.50	23.43	7.46	38.34	9.65	51.07	10.40	65.97	11.11	83.51	11.82	125.95	13.09	181.04	14.41	247.08	15.54	327.90	16.70
1.43	22.80	7.26	37.26	9.47	49.64	10.16	63.96	10.85	81.42	11.58	123.10	12.84	176.30	14.01	241.60	15.21	318.20	16.30
1.40	22.63	7.20	37.04	9.32	49.33	10.05	63.72	10.73	80.62	11.41	121.64	12.64	174.40	13.88	238.70	15.01	316.79	16.13
1.30	21.80	6.94	35.69	8.98	47.53	9.68	61.14	10.34	77.66	10.99	117.08	12.17	168.06	13.37	229.97	14.46	305.25	15.55
1.25	21.38	6.51	34.99	8.80	46.40	9.40	60.00	10.00	75.00	10.70	114.00	11.90	164.00	13.00	225.00	14.10	298.00	15.20
1.20	20.95	6.67	34.29	8.62	45.66	9.30	58.97	9.93	74.62	10.56	112.63	11.71	161.56	12.86	221.00	13.90	293.99	14.87
1.11	20.14	6.41	33.00	8.30	43.91	8.96	56.71	9.55	71.76	10.15	108.29	11.30	155.31	12.36	212.50	13.36	282.15	14.40
1.10	20.09	6.38	32.83	8.26	43.72	8.91	56.46	9.51	71.44	10.11	107.81	11.21	154.62	12.31	211.55	13.30	280.84	14.30
1.00	19.13	6.09	31.33	7.88	41.63	8.48	53.81	9.09	68.14	9.64	102.95	10.70	147.40	11.73	201.51	12.87	267.62	13.63
0.98	18.93	6.02	30.98	7.79	41.26	8.41	53.30	8.97	67.42	9.54	101.75	10.58	145.90	11.61	199.66	12.55	265.97	13.50
0.95	18.63	5.93	30.46	7.67	40.62	8.26	52.17	8.83	66.36	9.39	100.60	10.41	143.68	11.43	196.56	12.36	261.30	13.29
0.92	18.33	5.84	30.01	7.55	39.97	8.14	51.63	8.69	65.28	9.24	99.59	10.25	141.39	11.25	193.46	12.16	256.73	13.08
0.90	17.90	5.70	29.30	7.40	39.00	7.96	51.00	8.52	64.00	9.06	97.00	10.10	139.00	11.10	191.00	12.00	253.00	12.80
0.87	17.60	5.60	28.70	7.20	38.00	7.79	50.00	8.34	63.00	8.87	95.00	9.87	136.00	10.80	187.00	11.70	247.00	12.60
0.83	17.20	5.50	28.10	7.10	37.00	7.63	48.00	8.16	61.00	8.68	93.00	9.66	133.00	10.60	183.00	11.50	242.00	12.30
0.80	16.80	5.40	27.50	6.90	36.60	7.46	47.40	7.98	60.00	8.49	91.00	9.44	130.00	10.40	179.00	11.20	237.00	12.10
0.77	16.60	5.30	27.20	6.80	36.20	7.37	46.80	7.89	59.00	8.40	90.00	9.34	129.00	10.30	177.00	11.10	234.00	11.90
0.74	16.20	5.20	26.60	6.70	35.40	7.21	46.00	7.71	58.00	8.20	88.00	9.12	126.00	10.00	173.00	10.90	229.00	11.60
0.71	16.00	5.10	26.30	6.60	35.00	7.12	45.00	7.62	57.00	8.10	87.00	9.02	124.00	9.90	171.00	10.70	226.00	11.50
0.69	15.70	5.00	25.70	6.50	34.10	6.96	44.20	7.45	56.00	7.92	85.00	8.81	122.00	9.67	167.00	10.50	221.00	11.20
0.67	15.30	4.90	25.40	6.40	33.70	6.87	43.70	7.36	55.00	7.82	84.00	8.70	120.00	9.55	165.00	10.30	218.00	11.10
0.65	15.10	4.80	24.70	6.20	32.90	6.70	43.00	7.18	54.00	7.63	82.00	8.49	117.00	9.32	161.00	10.10	214.00	10.80
0.63	14.80	4.70	24.40	6.10	32.50	6.62	42.00	7.09	53.00	7.54	81.00	8.38	116.00	9.20	159.00	9.97	210.00	10.70
0.61	14.70	4.70	24.10	6.00	32.10	6.54	41.50	6.99	52.60	7.44	80.00	8.28	114.00	9.09	157.00	9.84	207.00	10.60
0.59	14.50	4.60	23.80	6.00	31.70	6.45	41.00	6.91	51.90	7.35	79.00	8.17	113.00	8.97	155.00	9.72	205.00	10.40
0.57	14.30	4.50	23.50	5.90	31.30	6.37	40.50	6.82	51.20	7.25	78.00	8.06	111.00	8.85	153.00	9.59	202.00	10.30
0.56	14.20	4.50	23.20	5.80	30.80	6.29	39.90	6.73	50.60	7.16	76.60	7.96	110.00	8.74	151.00	9.47	199.00	10.20
0.54	14.00	4.40	22.90	5.80	30.40	6.20	39.40	6.64	49.80	7.06	75.60	7.85	108.00	8.62	149.00	9.34	197.00	10.00
0.53	13.80	4.40	22.60	5.70	30.00	6.12	38.90	6.55	49.20	6.96	74.50	7.74	107.00	8.50	147.00	9.21	194.00	9.90
0.51	13.60	4.30	22.30	5.60	29.60	6.03	38.30	6.46	48.50	6.87	73.50	7.64	105.00	8.39	144.00	9.08	191.00	9.70
0.50	13.40	4.30	22.00	5.50	29.20	5.95	38.00	6.37	48.00	6.77	72.50	7.53	104.00	8.27	142.00	8.96	189.00	9.60
0.48	13.00	4.10	21.30	5.40	28.40	5.78	36.80	6.19	46.50	6.58	70.50	7.32	101.00	8.04	138.00	8.71	183.00	9.30
0.45	12.60	4.00	20.70	5.20	27.60	5.61	35.70	6.01	45.20	6.39	68.40	7.11	98.00	7.81	134.00	8.45	178.00	9.10
0.43	12.50	4.00	20.40	5.10	27.10	5.53	35.20	5.92	44.50	6.30	67.40	7.00	97.00	7.69	132.00	8.33	175.00	8.90
0.42	12.30	3.90	20.10	5.00	26.70	5.45	34.60	5.83	43.80	6.20	66.40	6.90	95.00	7.57	130.00	8.20	173.00	8.80
0.40	11.90	3.80	19.50	4.90	25.90	5.28	33.60	5.65	42.50	6.01	64.30	6.68	92.00	7.34	126.00	7.95	168.00	8.50
0.38	11.70	3.70	19.20	4.80	25.50	5.20	33.00	5.56	41.80	5.91	63.30	6.58	91.00	7.22	124.00	7.82	165.00	8.40
0.37	11.50	3.70	18.90	4.70	25.10	5.11	32.50	5.47	41.10	5.82	62.30	6.47	89.00	7.11	122.00	7.70	162.00	8.30
0.36	11.30	3.60	18.60	4.70	24.70	5.03	32.00	5.38	40.40	5.72	61.30	6.37	88.00	6.99	120.00	7.57	160.00	8.10
0.34	11.10	3.50	18.20	4.60	24.30	4.94	31.40	5.29	39.80	5.63	60.20	6.26	86.40	6.87	118.00	7.44	157.00	8.00
0.33	10.90	3.50	17.90	4.05	23.90	4.86	31.00	5.20	39.00	5.53	59.00	6.15	85.00	6.76	116.00	7.32	154.00	7.80
0.32	10.80	3.40	17.60	4.40	23.40	4.78	30.40	5.11	38.40	5.44	58.20	6.05	83.40	6.64	114.00	7.19	152.00	7.70
0.31	10.60	3.40	17.30	4.40	23.00	4.69	29.80	5.02	37.70	5.34	57.20	5.94	82.00	6.52	112.00	7.07	149.00	7.60
0.30	10.40	3.30	17.00	4.30	22.60	4.61	29.30	4.93	37.10	5.25	56.20	5.84	80.50	6.41	110.00	6.94	146.00	7.40
0.29	10.20	3.20	16.70	4.20	22.20	4.53	28.80	4.84	36.40	5.15	55.10	5.73	79.10	6.29	108.00	6.81	144.00	7.30
0.285	10.10	3.20	16.50	4.20	21.90	4.47	28.50	4.79	36.00	5.09	54.50	5.67	78.00	6.22	107.00	6.74	142.00	7.20
0.28	9.90	3.20	16.30	4.10	21.70	4.42	28.10	4.73	35.50	5.03	53.80	5.59	77.10	6.14	106.00	6.65	140.00	7.10
0.27	9.80	3.10	16.10	4.00	21.40	4.36	27.70	4.66	35.10	4.96	53.10	5.52	76.10	6.06	104.00	6.56	138.00	7.00
0.26	9.70	3.10	15.90	4.00	21.10	4.30	27.30	4.60	34.60	4.89	52.40	5.44	75.10	5.98	103.00	6.47	136.00	6.90
0.255	9.60	3.00	15.60	3.90	20.80	4.24	27.00	4.54	34.10	4.83	51.70	5.37	74.00	5.89	102.00	6.38	135.00	6.80
0.25	9.40	3.00	15.40	3.90	20.60	4.19	26.60	4.48	33.70	4.77	51.00	5.30	73.00	5.82	100.00	6.31	133.00	6.80
0.24	9.20	2.90	15.10	3.80	20.10	4.09	26.00	4.38	32.90	4.66	49.80	5.18	71.00	5.69	98.00	6.16	130.00	6.60
0.23	9.00	2.90	14.70	3.70	19.60	4.00	25.40	4.28	32.20	4.55	48.70	5.06	70.00	5.56	96.00	6.02	127.00	6.40
0.22	8.80	2.80	14.40	3.60	19.20	3.91	24.80	4.18	31.60	4.45	47.60	4.94	68.00	5.43	94.00	5.88	124.00	6.30
0.21	8.60	2.70	14.10	3.50	18.80	3.82	24.30	4.09	30.70	4.35	46.60	4.84	67.00	5.31	92.00	5.73	121.00	6.20
0.20	8.40	2.70	13.80	3.50	18.40	3.74	23.80	4.00	30.10	4.26	46.00	4.74	65.00	5.21	90.00	5.64	119.00	6.10
0.167	7.70	2.50	12.60	3.20	16.80	3.42	21.70	3.66	27.50	3.89	42.00	4.33	60.00	4.75	82.00	5.15	108.00	5.50
0.143	7.10	2.30	11.70	2.90	15.50	3.17	20.10	3.39	25.50	3.61	39.00	4.01	55.00	4.40	76.00	4.77	101.00	5.10
0.125	6.60	2.10	10.90	2.70	14.40	2.94	18.70	3.15	23.70	3.35	36.00	3.72	51.00	4.09	70.00	4.43	93.00	4.80
0.10	6.00	1.90	9.80	2.50	13.00	2.65	16.80	2.83	21.30	3.01	32.27	3.39	46.00	3.68	63.00	3.99	84.00	4.30

DISCHARGE AND VELOCITY FOR CIRCULAR VITRIFIED OR CONCRETE PIPES 6" TO 21"

MANNING FORMULA

BOROUGH OF SCARBOROUGH
WORKS DEPARTMENT
APRIL 1, 1970

$$Q = \frac{1.486 \cdot R^{2/3} \cdot S^{1/2} \cdot A}{n}$$

$$V = \frac{1.486 \cdot R^{2/3} \cdot S^{1/2}}{n}$$

"n" = .013

% GRADE	6" Q	6" V	8" Q	8" V	9" Q	9" V	10" Q	10" V	12" Q	12" V	15" Q	15" V	18" Q	18" V	21" Q	21" V
6.00	1.37	7.00	2.97	8.51	4.10	9.26	5.35	9.81	8.74	11.13	15.86	12.93	25.81	14.57	38.8	16.13
5.00	1.25	6.39	2.71	7.77	3.74	8.46	4.88	8.96	7.97	10.16	14.47	11.79	23.50	13.30	35.4	14.72
4.00	1.12	5.71	2.42	6.95	3.34	7.57	4.37	8.01	7.13	9.09	12.94	10.55	21.02	11.90	31.7	13.16
3.50	1.04	5.35	2.27	6.50	3.13	7.08	4.08	7.50	6.67	8.50	12.11	9.87	19.66	11.13	29.6	12.31
3.00	0.97	5.11	2.10	6.02	2.90	6.55	3.78	6.94	6.18	7.87	11.21	9.13	18.20	10.30	27.4	11.40
2.50	0.89	4.95	1.92	5.49	2.64	5.98	3.45	6.33	5.64	7.18	10.23	8.34	16.62	9.40	25.0	10.41
2.00	0.79	4.52	1.71	4.91	2.36	5.35	3.09	5.66	5.04	6.42	9.15	7.46	14.86	8.41	22.4	9.31
1.80	0.75	4.04	1.63	4.66	2.24	5.08	2.93	5.38	4.79	6.10	8.68	7.08	14.10	7.98	21.2	8.83
1.60	0.71	3.83	1.53	4.39	2.12	4.79	2.76	5.07	4.51	5.75	8.18	6.67	13.30	7.52	20.0	8.33
1.50	0.69	3.61	1.48	4.25	2.05	4.63	2.67	4.90	4.37	5.57	7.93	6.46	12.88	7.29	19.4	8.06
1.40	0.66	3.38	1.43	4.11	1.98	4.48	2.58	4.74	4.22	5.37	7.65	6.24	12.43	7.04	18.7	7.79
1.30	0.64	3.26	1.38	3.96	1.91	4.31	2.49	4.57	4.07	5.18	7.38	6.01	11.98	6.78	18.0	7.50
1.20	0.61	3.12	1.33	3.80	1.83	4.14	2.39	4.39	3.90	4.97	7.08	5.77	11.51	6.51	17.3	7.21
1.10	0.59	2.97	1.27	3.64	1.75	3.97	2.29	4.20	3.74	4.77	6.79	5.53	11.02	6.24	16.6	6.90
1.00	0.56	2.86	1.21	3.47	1.67	3.78	2.18	4.01	3.57	4.54	6.47	5.27	10.51	5.95	15.8	6.58
0.98			1.20	3.44	1.66	3.75	2.16	3.97	3.53	4.50	6.41	5.22	10.40	5.89	15.65	6.56
0.96			1.19	3.40	1.64	3.71	2.14	3.93	3.49	4.45	6.34	5.17	10.30	5.83	15.50	6.45
0.94			1.18	3.37	1.62	3.67	2.12	3.89	3.46	4.41	6.28	5.11	10.19	5.77	15.35	6.38
0.92			1.16	3.33	1.60	3.63	2.09	3.84	3.42	4.36	6.20	5.06	10.08	5.70	15.18	6.31
0.90			1.15	3.30	1.58	3.59	2.07	3.80	3.38	4.31	6.14	5.00	9.97	5.64	15.02	6.25
0.88			1.14	3.26	1.57	3.55	2.05	3.76	3.34	4.26	6.07	4.95	9.86	5.58	14.85	6.17
0.86			1.12	3.22	1.55	3.51	2.02	3.71	3.31	4.21	6.00	4.89	9.74	5.51	14.67	6.10
0.84			1.11	3.18	1.53	3.47	2.00	3.67	3.27	4.17	5.93	4.84	9.64	5.45	14.52	6.04
0.82			1.10	3.14	1.51	3.43	1.98	3.63	3.23	4.12	5.86	4.78	9.52	5.39	14.34	5.96
0.80			1.08	3.10	1.49	3.38	1.95	3.58	3.19	4.06	5.78	4.71	9.40	5.32	14.15	5.88
0.78			1.07	3.07	1.48	3.34	1.93	3.54	3.15	4.01	5.71	4.66	9.28	5.25	13.98	5.81
0.76			1.06	3.03	1.46	3.31	1.90	3.49	3.11	3.96	5.64	4.60	9.16	5.19	13.80	5.74
0.74			1.04	2.99	1.44	3.26	1.88	3.45	3.07	3.91	5.56	4.53	9.04	5.12	13.61	5.66
0.72			1.03	2.95	1.42	3.21	1.85	3.40	3.03	3.86	5.49	4.48	8.92	5.05	13.44	5.59
0.70			1.01	2.91	1.40	3.17	1.83	3.35	2.98	3.80	5.42	4.41	8.80	4.98	13.25	5.51
0.68					1.38	3.12	1.80	3.30	2.94	3.75	5.34	4.35	8.67	4.91	13.06	5.43
0.66					1.36	3.07	1.77	3.25	2.90	3.69	5.25	4.28	8.53	4.83	12.85	5.34
0.64					1.34	3.03	1.75	3.20	2.85	3.63	5.18	4.22	8.41	4.76	12.66	5.27
0.62					1.32	2.98	1.72	3.15	2.81	3.58	5.09	4.15	8.27	4.68	12.46	5.18
0.60					1.30	2.93	1.69	3.10	2.76	3.52	5.01	4.09	8.15	4.61	12.27	5.10
0.58					1.27	2.88	1.66	3.05	2.72	3.46	4.93	4.02	8.01	4.53	12.06	5.02
0.56					1.25	2.83	1.63	3.00	2.67	3.40	4.84	3.94	7.86	4.45	11.84	4.92
0.54					1.23	2.78	1.60	2.94	2.62	3.34	4.76	3.88	7.72	4.37	11.64	4.84
0.52					1.21	2.73	1.57	2.89	2.57	3.27	4.66	3.80	7.58	4.29	11.41	4.75
0.50					1.18	2.68	1.54	2.83	2.52	3.21	4.57	3.73	7.43	4.21	11.19	4.65
0.48							1.51	2.78	2.47	3.15	4.48	3.65	7.28	4.12	10.97	4.56
0.46							1.48	2.72	2.42	3.08	4.39	3.58	7.13	4.03	10.73	4.46
0.44							1.45	2.66	2.36	3.01	4.29	3.50	6.97	3.94	10.50	4.36
0.42							1.41	2.60	2.31	2.94	4.19	3.42	6.81	3.85	10.26	4.26
0.40							1.38	2.53	2.25	2.87	4.09	3.33	6.64	3.76	10.00	4.16
0.35									2.11	2.69	3.83	3.12	6.22	3.52	9.37	3.90
0.30									1.95	2.49	3.55	2.89	5.76	3.26	8.67	3.61
0.25									1.78	2.27	3.24	2.64	5.25	2.97	7.92	3.29
0.20									1.59	2.03	2.89	2.36	4.70	2.66	7.08	2.94

DISCHARGE AND VELOCITY FOR CIRCULAR VITRIFIED OR CONCRETE PIPES 24" TO 48"

MANNING FORMULA

BOROUGH OF SCARBOROUGH
WORKS DEPARTMENT
APRIL 1, 1970

$$Q = \frac{1.486 \cdot R^{2/3} \cdot S^{1/2} \cdot A}{n}$$

$$V = \frac{1.486 \cdot R^{2/3} \cdot S^{1/2}}{n}$$

"n" = .013

%	24"		27"		30"		33"		36"		39"		42"		48"	
GRADE	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V
6.00	55.5	17.6	75.9	19.1	100.5	20.5	129.9	21.9	163.3	23.1	203.3	24.5	246.3	25.6	351.7	28.0
5.00	50.6	16.1	69.2	17.4	91.7	18.7	118.6	20.0	149.1	21.1	185.6	22.4	224.9	23.4	321.1	25.5
4.00	45.2	14.4	61.9	15.6	82.0	16.7	106.1	17.9	133.3	18.9	166.0	20.0	201.2	20.9	287.2	22.8
3.50	42.3	13.5	57.9	14.6	76.7	15.6	99.2	16.7	124.7	17.6	155.3	18.7	188.2	19.6	268.7	21.4
3.00	39.2	12.5	53.6	13.5	71.3	14.5	91.8	15.5	115.5	16.3	143.8	17.3	174.2	18.4	248.7	19.8
2.50	35.8	11.4	48.8	12.3	64.8	13.2	83.6	14.2	105.4	14.9	131.2	15.8	159.0	16.5	227.0	18.1
2.00	32.0	10.2	43.8	11.0	58.0	11.8	75.0	12.6	94.3	13.3	117.4	14.2	144.2	14.8	203.1	16.1
1.80	30.4	9.66	41.5	10.4	55.0	11.2	71.2	12.0	89.5	12.7	111.4	13.4	135.0	14.0	192.7	15.3
1.60	28.6	9.11	39.2	9.84	51.9	10.6	67.1	11.3	84.3	11.9	105.0	12.7	127.3	13.2	181.7	14.4
1.50	27.7	8.82	37.9	9.53	50.2	10.2	65.0	10.9	81.7	11.6	101.7	12.3	123.2	12.8	175.9	14.0
1.40	26.8	8.52	36.6	9.20	48.5	9.90	62.7	10.6	78.9	11.2	98.2	11.8	119.0	12.1	169.9	13.5
1.30	25.8	8.21	35.3	8.87	46.8	9.52	60.5	10.2	76.0	10.8	94.6	11.4	114.7	11.9	163.7	13.0
1.20	24.8	7.88	33.9	8.52	44.9	9.15	58.1	9.78	73.0	10.3	90.1	10.9	110.2	11.5	157.2	12.5
1.10	23.7	7.55	32.5	8.16	43.0	8.76	55.6	9.37	69.9	9.89	87.1	10.5	105.5	11.0	150.6	12.0
1.00	22.6	7.20	31.0	7.78	41.0	8.35	53.0	8.93	66.7	9.43	83.0	10.0	100.6	10.5	143.6	11.4
0.98	22.4	7.13	30.7	7.70	40.6	8.27	52.5	8.84	66.0	9.34	82.2	9.91	99.6	10.4	142.2	11.3
0.96	22.2	7.06	30.3	7.62	40.2	8.18	52.0	8.75	65.3	9.24	81.3	9.81	98.6	10.3	140.7	11.2
0.94	21.9	6.98	30.0	7.55	39.8	8.10	51.4	8.66	64.7	9.15	80.5	9.71	97.6	10.2	139.3	11.1
0.92	21.7	6.90	29.7	7.46	39.3	8.01	50.9	8.56	63.9	9.04	79.6	9.60	96.5	10.0	137.7	11.0
0.90	21.5	6.83	29.4	7.38	38.9	7.93	50.3	8.47	63.3	8.95	78.8	9.50	95.5	9.93	136.3	10.8
0.88	21.2	6.75	29.0	7.30	38.5	7.83	49.8	8.38	62.5	8.85	77.9	9.39	94.4	9.81	134.7	10.7
0.86	21.0	6.67	28.7	7.21	38.0	7.74	49.2	8.28	61.8	8.74	76.9	9.28	93.3	9.70	133.1	10.6
0.84	20.7	6.60	28.4	7.13	37.6	7.66	48.6	8.19	61.1	8.65	76.1	9.18	92.3	9.59	131.7	10.5
0.82	20.5	6.52	28.0	7.05	37.2	7.57	48.0	8.09	60.4	8.54	75.2	9.07	91.1	9.48	130.1	10.3
0.80	20.2	6.44	27.7	6.95	36.7	7.47	47.4	7.98	59.6	8.43	74.2	8.95	89.9	9.35	128.4	10.2
0.78	20.0	6.36	27.3	6.87	36.2	7.37	46.8	7.88	58.9	8.33	73.3	8.84	88.8	9.24	126.8	10.1
0.76	19.7	6.28	27.0	6.78	35.8	7.28	46.2	7.78	58.1	8.22	72.4	8.73	88.7	9.12	125.2	9.96
0.74	19.5	6.19	26.6	6.69	35.3	7.18	45.6	7.68	57.3	8.11	71.4	8.61	86.5	9.00	123.5	9.82
0.72	19.2	6.11	26.3	6.60	34.8	7.09	45.0	7.58	56.6	8.01	70.5	8.50	85.4	8.88	121.9	9.70
0.70	18.9	6.03	25.9	6.51	34.3	6.99	44.4	7.47	55.8	7.89	69.5	8.38	84.2	8.76	120.2	9.56
0.68	18.7	5.94	25.5	6.42	33.8	6.89	43.8	7.37	55.0	7.78	68.5	8.26	83.0	8.63	118.5	9.42
0.66	18.4	5.85	25.1	6.32	33.3	6.78	43.1	7.25	54.1	7.66	67.4	8.13	81.7	8.49	116.6	9.27
0.64	18.1	5.76	24.8	6.22	32.8	6.68	42.4	7.14	53.3	7.54	66.4	8.01	80.5	8.37	114.9	9.14
0.62	17.8	5.67	24.4	6.12	32.3	6.57	41.7	7.03	52.5	7.42	65.3	7.88	79.2	8.23	113.0	8.99
0.60	17.5	5.58	24.0	6.03	31.8	6.47	41.1	6.92	51.7	7.31	64.3	7.76	78.0	8.11	111.3	8.85
0.58	17.2	5.49	23.6	5.93	31.2	6.36	40.4	6.80	50.8	7.19	63.3	7.63	76.7	7.97	109.4	8.70
0.56	16.9	5.38	23.1	5.82	30.7	6.25	39.7	6.68	49.9	7.05	62.1	7.49	75.2	7.82	107.4	8.54
0.54	16.6	5.29	22.7	5.72	30.1	6.14	39.0	6.56	49.0	6.93	61.0	7.36	73.9	7.69	105.5	8.39
0.52	16.3	5.19	22.3	5.61	29.6	6.02	38.2	6.44	48.1	6.80	59.9	7.22	72.5	7.54	103.5	8.23
0.50	16.0	5.09	21.9	5.50	29.0	5.90	37.5	6.31	47.1	6.67	58.7	7.08	71.1	7.40	101.5	8.07
0.48	15.7	4.99	21.5	5.39	28.4	5.79	36.8	6.19	46.2	6.53	57.5	6.94	69.7	7.25	99.5	7.91
0.46	15.4	4.88	21.0	5.27	27.8	5.66	36.0	6.05	45.2	6.39	56.3	6.79	68.2	7.09	97.4	7.74
0.44	15.0	4.77	20.5	5.16	27.2	5.54	35.2	5.92	44.2	6.25	55.0	6.64	66.7	6.93	95.2	7.57
0.42	14.6	4.66	20.1	5.04	26.6	5.41	34.4	5.79	43.2	6.11	53.8	6.49	65.2	6.78	93.0	7.40
0.40	14.3	4.55	19.6	4.92	25.9	5.28	33.5	5.64	42.1	5.96	52.5	6.33	63.6	6.61	90.8	7.22
0.35	13.4	4.26	18.3	4.61	24.3	4.94	31.4	5.29	39.5	5.58	49.1	5.93	59.6	6.19	85.0	6.76
0.30	12.4	3.95	17.0	4.26	22.5	4.58	29.1	4.89	36.5	5.17	45.5	5.49	55.1	5.73	78.7	6.26
0.25	11.3	3.60	15.5	3.89	20.5	4.18	26.5	4.46	33.3	4.71	41.5	5.01	50.3	5.23	71.8	5.71
0.20	10.1	3.22	13.8	3.48	18.3	3.73	23.7	3.99	29.8	4.21	37.1	4.47	45.0	4.68	64.2	5.10

D-5.2

DISCHARGE AND VELOCITY
FOR
CIRCULAR CONCRETE PIPES
54" TO 96"

MANNING FORMULA

$$Q = \frac{1.486 \cdot R^{2/3} \cdot S^{1/2} \cdot A}{n}$$

$$V = \frac{1.486 \cdot R^{2/3} \cdot S^{1/2}}{n}$$

BOROUGH OF SCARBOROUGH
WORKS DEPARTMENT
APRIL 1, 1970

"n" = .013

SIZE	54"		60"		66"		72"		78"		84"		90"		96"	
GRADE	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V
6.00	481.9	30.3	638.0	32.5	822.5	34.6	1037	36.7	1284	38.7	1565	40.7	1882	42.6	2234	44.4
5.00	439.8	27.7	582.0	29.7	750.6	31.6	946.7	33.5	1172	35.3	1428	37.1	1717	38.9	2039	40.6
4.00	393.4	24.7	520.8	26.5	671.4	28.3	846.8	30.0	1048	31.6	1278	33.2	1536	34.8	1824	36.3
3.50	368.0	23.1	487.2	24.8	628.1	26.4	792.2	28.0	980.8	29.6	1195	31.1	1437	32.5	1706	33.9
3.00	340.7	21.4	451.0	23.0	581.4	24.5	733.3	25.9	907.9	27.4	1106	28.8	1330	30.1	1579	31.4
2.50	311.0	19.6	411.7	21.0	530.7	22.3	669.4	23.7	828.8	25.0	1010	26.2	1214	27.5	1442	28.7
2.00	278.1	17.5	368.2	18.8	474.7	20.0	598.7	21.2	741.2	22.3	903.2	23.5	1086	24.6	1289	25.6
1.80	263.9	16.6	349.5	17.8	450.5	19.0	568.2	20.1	703.5	21.2	857.3	22.3	1031	23.3	1224	24.3
1.60	248.8	15.6	329.4	16.8	424.7	17.9	535.6	18.9	663.1	20.0	808.1	21.0	971.7	22.0	1154	22.9
1.50	241.0	15.2	319.0	16.3	411.2	17.3	518.7	18.4	642.1	19.4	782.5	20.3	940.9	21.3	1117	22.2
1.40	232.7	14.6	308.1	15.7	397.1	16.7	500.9	17.7	620.1	18.7	755.7	19.6	908.7	20.6	1079	21.5
1.30	224.2	14.1	296.9	15.1	382.7	16.1	482.7	17.1	597.6	18.0	728.2	18.9	875.6	19.8	1040	20.7
1.20	215.4	13.5	285.1	14.5	367.6	15.5	463.6	16.4	574.0	17.3	699.5	18.2	841.1	19.0	998.5	19.9
1.10	206.3	13.0	273.2	13.9	352.1	14.8	444.1	15.7	549.9	16.6	670.1	17.4	805.7	18.2	956.5	19.0
1.00	196.7	12.4	260.4	13.3	335.7	14.1	423.4	15.0	524.2	15.8	638.8	16.6	768.1	17.4	911.9	18.1
0.98	194.7	12.2	257.8	13.1	332.3	14.0	419.2	14.8	519.0	15.6	632.4	16.4	760.4	17.2	902.8	18.0
0.96	192.7	12.1	255.2	13.0	329.0	13.8	414.9	14.7	513.7	15.5	626.0	16.3	752.7	17.0	893.7	17.8
0.94	190.8	12.0	252.6	12.9	325.6	13.7	410.7	14.5	508.5	15.3	619.6	16.1	745.1	16.9	884.5	17.6
0.92	188.6	11.9	249.7	12.7	321.9	13.6	406.0	14.4	502.7	15.2	612.6	15.9	736.6	16.7	874.5	17.4
0.90	186.7	11.7	247.1	12.6	318.6	13.4	401.8	14.2	497.5	15.0	606.2	15.8	728.9	16.5	865.4	17.2
0.88	184.5	11.6	244.3	12.4	315.2	13.3	397.1	14.1	491.7	14.8	599.2	15.6	720.5	16.3	855.4	17.0
0.86	182.3	11.5	241.4	12.3	311.2	13.1	392.5	13.9	485.9	14.6	592.2	15.4	712.0	16.1	845.3	16.8
0.84	180.4	11.3	238.8	12.2	307.8	13.0	388.3	13.7	480.7	14.5	585.8	15.2	704.3	15.9	836.2	16.6
0.82	178.2	11.2	235.9	12.0	304.1	12.8	383.6	13.6	474.9	14.3	578.8	15.0	695.9	15.7	826.2	16.4
0.80	175.8	11.0	232.8	11.9	300.1	12.6	378.5	13.4	468.6	14.1	571.1	14.8	686.7	15.5	815.2	16.2
0.78	173.7	10.9	229.9	11.7	296.4	12.5	373.9	13.2	462.9	13.9	564.1	14.7	678.2	15.4	805.2	16.0
0.76	171.5	10.8	227.1	11.6	292.7	12.3	369.2	13.1	457.1	13.8	557.0	14.5	669.8	15.2	795.2	15.8
0.74	169.2	10.6	223.9	11.4	288.7	12.1	364.1	12.9	450.8	13.6	549.4	14.3	660.6	15.0	784.2	15.6
0.72	167.0	10.5	221.1	11.3	285.0	12.0	359.5	12.7	445.0	13.4	542.3	14.1	652.1	14.8	774.2	15.4
0.70	164.6	10.4	218.0	11.1	281.0	11.8	354.4	12.5	438.8	13.2	534.7	13.9	642.9	14.6	763.3	15.2
0.68	162.3	10.2	214.8	10.9	277.0	11.7	349.3	12.4	432.5	13.0	527.0	13.7	633.7	14.3	752.3	15.0
0.66	159.7	10.0	211.4	10.8	272.6	11.5	343.8	12.2	425.7	12.8	518.7	13.5	623.7	14.1	740.5	14.7
0.64	157.4	9.90	208.3	10.6	268.6	11.3	338.7	12.0	419.4	12.6	511.0	13.3	614.5	13.9	729.5	14.5
0.62	154.8	9.74	204.9	10.4	264.2	11.1	333.2	11.8	412.5	12.4	502.7	13.1	604.5	13.7	717.7	14.3
0.60	152.4	9.59	201.8	10.3	260.2	11.0	328.1	11.6	406.3	12.2	495.1	12.9	595.3	13.5	706.7	14.1
0.58	149.9	9.43	198.4	10.1	255.8	10.8	322.6	11.4	399.4	12.0	486.8	12.6	585.3	13.3	694.9	13.8
0.56	147.1	9.25	194.8	9.93	251.1	10.6	316.7	11.2	392.1	11.8	477.8	12.4	574.5	13.0	682.1	13.6
0.54	144.6	9.09	191.4	9.75	246.7	10.4	311.2	11.0	385.3	11.6	469.5	12.2	564.6	12.8	670.2	13.3
0.52	141.8	8.92	187.7	9.57	242.0	10.2	305.3	10.8	377.9	11.4	460.6	12.0	553.8	12.5	657.5	13.1
0.50	139.1	8.75	184.1	9.38	237.3	10.0	299.3	10.6	370.6	11.2	451.6	11.7	543.0	12.3	644.7	12.8
0.48	136.3	8.57	180.5	9.20	232.6	9.79	293.4	10.4	363.3	10.9	442.7	11.5	532.3	12.1	631.9	12.6
0.46	133.4	8.39	176.6	9.00	227.6	9.58	287.1	10.2	355.4	10.7	433.1	11.3	520.8	11.8	618.3	12.3
0.44	130.4	8.20	172.6	8.80	222.6	9.37	280.7	9.93	347.5	10.5	423.5	11.0	509.3	11.5	604.6	12.0
0.42	127.5	8.02	168.7	8.60	217.5	9.16	274.4	9.71	339.7	10.2	413.9	10.8	497.7	11.3	590.9	11.8
0.40	124.3	7.82	164.6	8.39	212.2	8.93	267.6	9.47	331.3	10.0	403.7	10.5	485.4	11.0	576.3	11.5
0.35	116.4	7.32	154.2	7.86	198.0	8.36	250.7	8.87	310.3	9.35	378.2	9.80	454.7	10.3	539.8	10.7
0.30	107.8	6.78	142.7	7.27	184.0	7.74	232.0	8.21	287.3	8.66	350.0	9.10	420.9	9.52	499.7	9.94
0.25	98.4	6.19	130.2	6.64	167.9	7.07	211.7	7.49	262.1	7.90	319.4	8.30	384.1	8.70	456.0	9.07
0.20	87.9	5.53	116.4	5.93	150.1	6.32	189.3	6.70	234.3	7.06	285.5	7.40	343.3	7.77	407.6	8.11

VELOCITY AND CAPACITY CALCULATIONS

MANNING FORMULA

$$Q = \frac{1.486 \cdot R^{2/3} \cdot S^{1/2} \cdot A}{n}$$

$$V = \frac{1.486 \cdot R^{2/3} \cdot S^{1/2}}{n}$$

CIRCULAR CONCRETE OR VITRIFIED SEWER PIPES
6" TO 120" FLOWING FULL
$n = .013$

BOROUGH OF SCARBOROUGH
WORKS DEPARTMENT
APRIL 1, 1970

TABLE 1

Diam. Inches	"A" Area	"R" M.H.D.	$R^{2/3}$	$\frac{1.486 \cdot R^{2/3} \cdot A}{.013}$ FOR CAPACITY	$\frac{1.486 \cdot R^{2/3}}{.013}$ FOR VELOCITY
6	.196	.125	.250	5.60	28.57
8	.349	.167	.303	12.12	34.73
9	.442	.187	.327	16.72	37.83
10	.545	.208	.351	21.83	40.06
12	.785	.250	.397	35.66	45.43
15	1.227	.312	.461	64.70	52.73
18	1.767	.375	.520	105.10	59.48
21	2.405	.437	.576	158.30	65.82
24	3.142	.500	.630	226.20	71.99
27	3.976	.562	.681	309.60	77.79
30	4.909	.625	.731	410.10	83.52
33	5.940	.687	.779	530.40	89.30
36	7.069	.750	.825	666.60	94.30
39	8.290	.821	.877	830.10	100.10
42	9.621	.875	.915	1006.00	104.60
48	12.566	1.000	1.000	1436.00	114.20
54	15.904	1.125	1.082	1967.00	123.70
60	19.635	1.250	1.160	2604.00	132.70
66	23.758	1.375	1.236	3357.00	141.30
72	28.274	1.500	1.310	4234.00	149.80
78	33.183	1.625	1.382	5242.00	158.00
84	38.485	1.750	1.452	6388.00	166.00
90	44.179	1.875	1.521	7681.00	173.90
96	50.266	2.000	1.587	9119.00	181.40
102	56.745	2.125	1.653	10,722.00	188.90
108	63.617	2.250	1.717	12,486.00	196.30
114	70.882	2.375	1.780	14,422.00	203.50
120	78.540	2.500	1.842	16,537.00	210.60

TABLE 2

% SLOPE	$S^{1/2}$	% SLOPE	$S^{1/2}$	% SLOPE	$S^{1/2}$	% SLOPE	$S^{1/2}$	% SLOPE	$S^{1/2}$
25.0	.5000	2.95	.1718	1.45	.1204	.79	.0889	.49	.0700
20.0	.4472	2.90	.1703	1.40	.1183	.78	.0883	.48	.0693
15.0	.3873	2.85	.1688	1.35	.1162	.77	.0878	.47	.0686
14.0	.3742	2.80	.1673	1.30	.1140	.76	.0872	.46	.0678
13.0	.3606	2.75	.1658	1.25	.1118	.75	.0866	.45	.0671
12.0	.3464	2.70	.1643	1.20	.1095	.74	.0860	.44	.0663
11.0	.3317	2.65	.1628	1.15	.1072	.73	.0854	.43	.0656
10.0	.3162	2.60	.1612	1.10	.1049	.72	.0849	.42	.0648
9.0	.3000	2.55	.1597	1.05	.1025	.71	.0843	.41	.0640
8.0	.2828	2.50	.1581	1.00	.1000	.70	.0837	.40	.0632
7.5	.2739	2.45	.1565	.99	.0995	.69	.0831	.39	.0625
7.0	.2646	2.40	.1549	.98	.0990	.68	.0825	.38	.0616
6.5	.2550	2.35	.1533	.97	.0985	.67	.0819	.37	.0608
6.0	.2449	2.30	.1517	.96	.0980	.66	.0812	.36	.0600
5.5	.2345	2.25	.1500	.95	.0975	.65	.0806	.35	.0592
5.0	.2236	2.20	.1483	.94	.0970	.64	.0800	.34	.0583
4.75	.2179	2.15	.1466	.93	.0964	.63	.0794	.33	.0574
4.50	.2121	2.10	.1449	.92	.0959	.62	.0787	.32	.0566
4.25	.2062	2.05	.1432	.91	.0954	.61	.0781	.31	.0557
4.00	.2000	2.00	.1414	.90	.0949	.60	.0775	.30	.0548
3.90	.1975	1.95	.1396	.89	.0943	.59	.0768	.29	.0539
3.80	.1949	1.90	.1378	.88	.0938	.58	.0762	.28	.0529
3.70	.1924	1.85	.1360	.87	.0933	.57	.0755	.27	.0520
3.60	.1897	1.80	.1342	.86	.0927	.56	.0748	.26	.0510
3.50	.1871	1.75	.1323	.85	.0922	.55	.0742	.25	.0500
3.40	.1844	1.70	.1304	.84	.0917	.54	.0735	.24	.0490
3.30	.1817	1.65	.1285	.83	.0911	.53	.0728	.23	.0480
3.20	.1789	1.60	.1265	.82	.0906	.52	.0721	.22	.0469
3.10	.1761	1.55	.1245	.81	.0900	.51	.0714	.21	.0458
3.00	.1732	1.50	.1225	.80	.0894	.50	.0707	.20	.0447

D-5.4

DISCHARGE AND VELOCITY FOR CIRCULAR ASBESTOS CEMENT PIPES

10" TO 24"

MANNING FORMULA

BOROUGH OF SCARBOROUGH
WORKS DEPARTMENT
APRIL 1, 1970

$$Q = \frac{1.486 \cdot R^{2/3} \cdot S^{1/2} \cdot A}{n}$$

$$V = \frac{1.486 \cdot R^{2/3} \cdot S^{1/2}}{n}$$

"n" = .010

GRADE %	10"		12"		14"		16"		18"		20"		24"	
	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V
1.00	2.84	5.21	4.64	5.91	6.90	6.51	10.10	7.16	13.70	7.73	17.80	8.22	29.4	9.36
.98	2.81	5.15	4.59	5.85	6.83	6.44	10.00	7.09	13.50	7.65	17.60	8.14	29.1	9.27
.96	2.78	5.10	4.54	5.79	6.76	6.38	9.90	7.02	13.40	7.58	17.40	8.05	28.8	9.17
.94	2.75	5.05	4.50	5.73	6.69	6.31	9.80	6.95	13.30	7.50	17.20	7.97	28.5	9.08
.92	2.72	4.99	4.45	5.66	6.62	6.24	9.69	6.87	13.10	7.41	17.00	7.88	28.2	8.98
.90	2.69	4.94	4.40	5.60	6.55	6.18	9.58	6.80	13.00	7.34	16.80	7.80	27.9	8.88
.88	2.66	4.88	4.35	5.54	6.47	6.11	9.47	6.72	12.80	7.25	16.60	7.71	27.6	8.78
.86	2.63	4.83	4.30	5.47	6.40	6.03	9.36	6.64	12.70	7.17	16.50	7.62	27.3	8.68
.84	2.60	4.77	4.25	5.42	6.33	5.97	9.26	6.57	12.50	7.09	16.30	7.54	27.0	8.58
.82	2.57	4.72	4.20	5.35	6.25	5.90	9.15	6.49	12.40	7.00	16.10	7.45	26.6	8.48
.80	2.54	4.66	4.14	5.28	6.17	5.82	9.03	6.40	12.20	6.91	15.90	7.35	26.3	8.37
.78	2.51	4.60	4.09	5.21	6.09	5.75	8.92	6.32	12.10	6.83	15.70	7.26	26.0	8.26
.76	2.47	4.54	4.04	5.15	6.02	5.68	8.81	6.25	11.90	6.74	15.50	7.17	25.6	8.16
.74	2.44	4.48	3.99	5.08	5.93	5.60	8.69	6.16	11.70	6.65	15.30	7.07	25.3	8.05
.72	2.41	4.42	3.94	5.01	5.86	5.53	8.57	6.08	11.60	6.56	15.10	6.98	25.0	7.95
.70	2.38	4.36	3.88	4.94	5.78	5.45	8.45	6.00	11.40	6.47	14.90	6.88	24.6	7.83
.68	2.34	4.30	3.82	4.87	5.69	5.37	8.33	5.91	11.30	6.38	14.60	6.78	24.3	7.72
.66	2.30	4.23	3.76	4.80	5.60	5.29	8.20	5.82	11.10	6.28	14.40	6.67	23.9	7.60
.64	2.27	4.17	3.71	4.72	5.52	5.21	8.08	5.73	10.90	6.18	14.20	6.57	23.5	7.49
.62	2.23	4.10	3.65	4.65	5.43	5.12	7.95	5.64	10.80	6.08	14.00	6.47	23.1	7.37
.60	2.20	4.04	3.59	4.58	5.35	5.04	7.83	5.55	10.60	5.99	13.80	6.37	22.8	7.25
.58	2.16	3.97	3.53	4.50	5.26	4.96	7.70	5.46	10.40	5.89	13.50	6.26	22.4	7.13
.56	2.12	3.89	3.47	4.42	5.16	4.87	7.55	5.36	10.20	5.78	13.30	6.15	22.0	7.00
.54	2.09	3.83	3.41	4.34	5.07	4.78	7.42	5.26	10.00	5.68	13.00	6.04	21.6	6.88
.52	2.05	3.75	3.34	4.26	4.97	4.69	7.28	5.16	9.85	5.57	12.80	5.93	21.2	6.75
.50	2.01	3.68	3.28	4.18	4.88	4.60	7.14	5.06	9.66	5.47	12.50	5.81	20.8	6.62
.48	1.97	3.61	3.21	4.09	4.78	4.51	7.00	4.96	9.47	5.36	12.30	5.70	20.4	6.49
.46	1.92	3.53	3.14	4.00	4.68	4.41	6.85	4.86	9.26	5.24	12.00	5.57	19.9	6.35
.44	1.88	3.45	3.07	3.92	4.57	4.32	6.70	4.75	9.06	5.13	11.80	5.45	19.5	6.21
.42	1.84	3.37	3.00	3.83	4.47	4.22	6.54	4.64	8.85	5.01	11.50	5.33	19.1	6.07
.40	1.79	3.29	2.93	3.73	4.36	4.11	6.38	4.53	8.63	4.89	11.20	5.19	18.6	5.92
.38	1.75	3.21	2.86	3.64	4.28	4.01	6.22	4.41	8.41	4.76	10.90	5.06	18.1	5.77
.36	1.70	3.12	2.78	3.54	4.14	3.91	6.06	4.30	8.20	4.64	10.60	4.93	17.6	5.62
.34	1.65	3.04	2.70	3.44	4.02	3.79	5.89	4.18	7.96	4.51	10.30	4.79	17.1	5.46
.32	1.61	2.95	2.63	3.34	3.91	3.68	5.72	4.05	7.73	4.28	10.00	4.65	16.6	5.30
.30	1.56	2.85	2.54	3.24	3.78	3.57	5.53	3.93	7.49	4.24	9.73	4.50	16.1	5.13
.28	1.50	2.75	2.45	3.12	3.65	3.44	5.34	3.79	7.23	4.09	9.39	4.35	15.6	4.95
.26	1.45	2.65	2.36	3.01	3.52	3.32	5.15	3.65	6.97	3.94	9.05	4.19	15.0	4.77
.24	1.39	2.55	2.27	2.89	3.38	3.19	4.95	3.51	6.69	3.79	8.70	4.03	14..4	4.59
.22	1.33	2.44	2.17	2.77	3.24	3.05	4.74	3.36	6.41	3.63	8.32	3.85	13.8	4.39
.20	1.27	2.33	2.07	2.64	3.08	2.91	4.51	3.20	6.11	3.46	7.93	3.67	13.1	4.18

DISCHARGE AND VELOCITY FOR CIRCULAR CORRUGATED METAL PIPES 12" TO 48"

MANNING FORMULA

$$Q = \frac{1.486 \cdot R^{2/3} \cdot S^{1/2} \cdot A}{n}$$

$$V = \frac{1.486 \cdot R^{2/3} \cdot S^{1/2}}{n}$$

BOROUGH OF SCARBOROUGH
WORKS DEPARTMENT
APRIL 1, 1970

"n" = .024

%	12"		15"		18"		24"		30"		36"		42"		48"	
GRADE	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V
6.00	4.73	6.03	8.58	6.99	13.93	7.89	30.00	9.53	54.40	11.10	88.40	12.50	133.50	13.90	190.50	15.20
5.00	4.32	5.50	7.83	6.38	12.72	7.20	27.40	8.70	49.70	10.10	80.70	11.40	121.90	12.70	174.00	13.80
4.00	3.86	4.92	7.01	5.71	11.38	6.44	24.50	7.78	44.40	9.05	72.20	10.20	109.00	11.30	155.60	12.40
3.50	3.61	4.60	6.56	5.34	10.65	6.02	22.90	7.28	41.60	8.47	67.60	9.54	102.00	10.60	145.60	11.60
3.00	3.35	4.27	6.07	4.94	9.86	5.58	21.20	6.74	38.50	7.84	62.50	8.83	94.40	9.82	134.70	10.70
2.50	3.05	3.89	5.54	4.51	9.00	5.09	19.40	6.15	35.10	7.15	57.10	8.06	86.20	8.96	123.00	9.79
2.00	2.73	3.48	4.95	4.04	8.05	4.55	17.30	5.50	31.40	6.40	51.10	7.21	77.10	8.02	110.00	8.75
1.80	2.59	3.30	4.70	3.83	7.64	4.32	16.40	5.22	29.80	6.07	48.50	6.84	73.10	7.61	104.40	8.31
1.60	2.44	3.11	4.43	3.61	7.20	4.07	15.50	4.92	28.10	5.72	45.70	6.45	68.90	7.17	98.40	7.83
1.50	2.37	3.01	4.29	3.50	6.97	3.94	15.00	4.77	27.20	5.54	44.20	6.25	66.80	6.95	95.30	7.58
1.40	2.29	2.91	4.15	3.38	6.73	3.81	14.50	4.60	26.30	5.35	42.70	6.03	64.50	6.71	92.00	7.32
1.30	2.20	2.81	3.99	3.25	6.54	3.67	14.00	4.43	25.30	5.16	41.20	5.81	62.10	6.46	88.70	7.06
1.20	2.12	2.69	3.84	3.13	6.23	3.53	13.40	4.26	24.30	4.95	39.50	5.58	59.70	6.21	85.20	6.78
1.10	2.03	2.58	3.68	2.99	5.97	3.38	12.90	4.08	23.30	4.75	37.90	5.35	57.20	5.95	81.60	6.49
1.00	1.93	2.46	3.50	2.86	5.69	3.22	12.30	3.89	22.20	4.53	36.10	5.10	54.50	5.67	77.80	6.19
0.98	1.91	2.44	3.47	2.83	5.63	3.19	12.10	3.85	22.00	4.48	35.70	5.05	54.00	5.61	77.00	6.12
0.96	1.89	2.41	3.43	2.80	5.58	3.16	12.00	3.81	21.80	4.43	35.40	5.00	53.40	5.56	76.20	6.07
0.94	1.87	2.39	3.40	2.77	5.52	3.12	11.90	3.77	21.60	4.39	35.00	4.95	52.90	5.50	75.50	6.00
0.92	1.85	2.36	3.36	2.74	5.46	3.09	11.70	3.73	21.30	4.34	34.60	4.89	52.30	5.44	74.60	5.93
0.90	1.83	2.34	3.33	2.71	5.40	3.06	11.60	3.69	21.10	4.29	34.30	4.84	51.70	5.38	73.60	5.87
0.88	1.81	2.31	3.29	2.68	5.34	3.02	11.50	3.65	20.80	4.24	33.90	4.78	51.10	5.32	73.00	5.81
0.86	1.79	2.28	3.25	2.65	5.27	2.98	11.40	3.61	20.60	4.19	33.50	4.73	50.50	5.26	72.10	5.74
0.84	1.77	2.26	3.21	2.62	5.22	2.95	11.20	3.57	20.40	4.15	33.10	4.68	50.00	5.20	71.30	5.68
0.82	1.75	2.23	3.17	2.59	5.16	2.92	11.10	3.52	20.10	4.10	32.70	4.62	49.40	5.14	70.50	5.51
0.80	1.73	2.20	3.13	2.56	5.09	2.88	11.00	3.48	19.90	4.05	32.30	4.56	48.70	5.07	69.60	5.53
0.78	1.71	2.17	3.09	2.52	5.02	2.84	10.80	3.43	19.60	4.00	31.90	4.50	48.10	5.01	68.70	5.47
0.76	1.68	2.15	3.06	2.49	4.96	2.81	10.70	3.39	19.40	3.95	31.50	4.45	47.50	4.94	67.80	5.40
0.74	1.66	2.12	3.01	2.46	4.89	2.77	10.50	3.35	19.10	3.89	31.10	4.39	46.90	4.88	66.90	5.32
0.72	1.64	2.09	2.97	2.42	4.83	2.73	10.40	3.30	18.90	3.84	30.70	4.33	46.30	4.81	66.10	5.26
0.70	1.62	2.06	2.93	2.39	4.76	2.70	10.30	3.26	18.60	3.79	30.20	4.27	45.60	4.75	65.10	5.18
0.68	1.59	2.03	2.89	2.36	4.69	2.66	10.10	3.21	18.30	3.73	29.80	4.21	45.00	4.68	64.20	5.11
0.66	1.57	2.00	2.85	2.32	4.62	2.61	9.95	3.16	18.00	3.67	29.30	4.14	44.30	4.60	63.20	5.03
0.64	1.55	1.97	2.80	2.28	4.55	2.58	9.80	3.11	17.80	3.62	28.90	4.08	43.60	4.54	62.20	4.95
0.62	1.52	1.94	2.76	2.25	4.48	2.53	9.64	3.06	17.50	3.56	28.40	4.01	42.90	4.46	61.20	4.87
0.60			2.72	2.21	4.41	2.50	9.49	3.01	17.20	3.51	28.00	3.95	42.20	4.39	60.30	4.80
0.58			2.67	2.18	4.34	2.45	9.33	2.96	16.90	3.45	27.50	3.89	41.50	4.32	59.30	4.72
0.56			2.62	2.14	4.26	2.41	9.16	2.91	16.60	3.38	27.00	3.81	40.80	4.24	58.20	4.63
0.54			2.57	2.10	4.18	2.37	9.00	2.86	16.30	3.33	26.50	3.75	40.10	4.17	57.20	4.55
0.52			2.53	2.06	4.10	2.32	8.83	2.80	16.00	3.26	26.00	3.68	39.30	4.09	56.10	4.46
0.50			2.48	2.02	4.02	2.28	8.66	2.75	15.70	3.20	25.50	3.61	38.50	4.01	55.00	4.38
0.48			2.43	1.88	3.94	2.23	8.49	2.70	15.40	3.14	25.00	3.53	37.80	3.93	53.90	4.29
0.46			2.38	1.94	3.86	2.18	8.31	2.64	15.10	3.07	24.50	3.46	37.00	3.84	52.70	4.20
0.44					3.77	2.13	8.12	2.58	14.70	3.00	23.90	3.38	36.10	3.76	51.60	4.10
0.42					3.69	2.09	7.94	2.52	14.40	2.93	23.40	3.30	35.30	3.67	50.40	4.01
0.40					3.60	2.04	7.74	2.46	14.00	2.86	22.80	3.22	34.40	3.58	49.20	3.91
0.35					3.37	1.91	7.25	2.30	13.20	2.68	21.40	3.02	32.30	3.36	46.10	3.66
0.30					3.12	1.76	6.71	2.13	12.20	2.48	19.80	2.79	29.90	3.11	42.60	3.39
0.25					2.85	1.61	6.13	1.95	11.10	2.26	18.10	2.55	27.30	2.84	38.90	3.10
0.20					2.54	1.44	5.48	1.74	9.93	2.02	16.10	2.28	24.40	2.53	34.80	2.77

DISCHARGE AND VELOCITY FOR CIRCULAR CORRUGATED METAL PIPES 54" TO 96"

MANNING FORMULA

BOROUGH OF SCARBOROUGH
WORKS DEPARTMENT
APRIL 1, 1970

$$Q = \frac{1.486 \cdot R^{2/3} \cdot S^{1/2} \cdot A}{n}$$

$$V = \frac{1.486 \cdot R^{2/3} \cdot S^{1/2}}{n}$$

"n" = .024

%	54"		60"		66"		72"		78"		84"		90"		96"	
GRADE	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V
6.00	260.80	16.40	360.40	18.40	464.60	19.56	585.00	20.70	725.60	21.90	847.40	22.00	1019.00	23.00	1210.00	24.00
5.00	238.10	15.00	329.10	16.60	424.20	17.86	535.00	18.90	662.50	20.00	773.70	20.10	930.40	21.00	1104.00	22.00
4.00	213.00	13.40	294.40	15.00	479.40	15.97	478.60	16.90	592.60	17.90	692.00	18.00	832.20	18.80	987.80	19.60
3.50	199.30	12.50	275.40	14.00	354.90	14.94	447.70	15.80	554.40	16.70	647.40	16.80	778.50	17.60	924.10	18.40
3.00	184.40	11.60	254.90	13.00	328.60	13.83	414.50	14.70	513.20	15.50	599.30	15.60	720.70	16.30	855.40	17.00
2.50	168.40	10.60	232.70	11.80	300.00	12.63	379.10	13.40	463.50	14.10	548.10	14.20	659.10	14.90	780.90	15.60
2.00	150.60	9.47	208.10	10.60	268.20	11.29	338.40	12.00	419.00	12.60	489.20	12.70	588.40	13.30	698.40	13.90
1.80	142.90	8.99	197.50	10.00	254.60	10.71	321.10	11.40	397.60	12.00	464.30	12.10	558.40	12.60	662.80	13.20
1.60	134.70	8.47	186.20	9.48	240.00	10.10	302.70	10.70	374.80	11.30	437.70	11.40	526.30	11.90	624.80	12.40
1.50	130.50	8.21	180.30	9.18	232.40	9.78	293.10	10.40	363.00	10.90	423.90	11.00	509.70	11.50	605.00	12.00
1.40	126.00	7.92	174.10	8.87	224.40	9.45	283.10	10.00	350.50	10.60	409.30	10.60	492.20	11.10	584.30	11.60
1.30	121.40	7.64	167.80	8.54	216.30	9.10	272.80	9.65	337.80	10.20	394.40	10.20	474.40	10.70	563.00	11.20
1.20	116.60	7.33	161.10	8.21	207.70	8.74	262.00	9.27	324.40	9.78	378.90	9.84	455.60	10.30	540.80	10.80
1.10	111.70	7.03	154.40	7.86	199.00	8.38	251.00	8.88	310.80	9.37	363.00	9.43	436.50	9.87	518.10	10.30
1.00	106.50	6.70	147.20	7.50	189.70	7.98	239.30	8.46	296.30	8.93	346.00	8.99	416.10	9.41	493.90	9.82
0.98	105.40	5.63	145.70	7.42	187.80	7.90	236.90	8.38	293.30	8.84	342.50	8.90	411.90	9.32	489.00	9.72
0.96	104.40	6.56	144.30	7.35	185.90	7.82	234.50	8.29	290.40	8.75	339.10	8.81	407.80	9.22	484.00	9.62
0.94	103.30	6.50	142.80	7.27	184.00	7.74	232.10	8.21	287.40	8.66	335.60	8.72	403.60	9.13	479.10	9.53
0.92	102.10	6.42	141.20	7.19	181.90	7.66	229.50	8.12	284.20	8.56	331.80	8.62	399.00	9.02	473.70	9.42
0.90	101.10	6.36	139.70	7.11	180.00	7.58	227.10	8.03	281.20	8.47	328.40	8.53	394.90	8.93	468.70	9.32
0.88	99.90	6.28	138.10	7.03	177.90	7.49	224.50	7.94	277.90	8.38	324.50	8.43	390.30	8.83	463.30	9.21
0.86	98.70	6.21	136.50	6.95	175.90	7.40	221.80	7.85	274.70	8.28	320.70	8.33	385.70	8.72	457.80	9.10
0.84	97.70	6.14	135.00	6.87	173.90	7.32	219.40	7.76	271.70	8.19	317.30	8.24	381.60	8.63	452.90	9.00
0.82	96.50	6.07	133.40	6.79	171.90	7.23	216.80	7.67	268.40	8.09	313.50	8.14	377.00	8.53	447.50	8.90
0.80	95.20	5.99	131.60	6.70	169.50	7.14	213.90	7.57	264.90	7.98	309.30	8.04	372.00	8.41	441.50	8.78
0.78	94.00	5.91	130.00	6.62	167.50	7.05	211.30	7.47	261.60	7.89	305.50	7.94	367.40	8.31	436.10	8.67
0.76	92.90	5.84	128.40	6.54	165.40	6.96	208.70	7.38	258.40	7.79	301.70	7.84	362.80	8.21	430.70	8.56
0.74	91.60	5.76	126.60	6.45	163.10	6.87	205.80	7.28	254.80	7.68	297.60	7.73	357.80	8.09	424.80	8.45
0.72	90.40	5.69	125.00	6.36	161.10	6.78	203.20	7.19	251.60	7.57	293.80	7.63	353.20	7.99	418.80	8.31
0.70	89.10	5.61	123.20	6.27	158.80	6.68	200.30	7.08	248.00	7.47	289.60	7.52	348.30	7.88	413.40	8.22
0.68	87.90	5.53	121.40	6.18	156.50	6.58	197.40	6.98	244.40	7.37	285.50	7.42	343.30	7.76	407.50	8.10
0.66	86.50	5.44	119.50	6.09	154.00	6.48	194.30	6.87	240.60	7.25	281.00	7.30	337.90	7.64	401.00	7.97
0.64	85.20	5.36	117.80	6.00	151.80	6.39	191.40	6.77	237.00	7.14	276.80	7.19	332.90	7.53	395.10	7.86
0.62	83.80	5.27	115.80	5.90	149.30	6.28	188.30	6.66	233.20	7.03	272.30	7.08	327.50	7.41	388.70	7.73
0.60	82.50	5.19	114.10	5.81	147.00	6.19	185.50	6.56	229.60	6.92	268.20	6.97	322.50	7.29	382.80	7.61
0.58	81.20	5.10	112.20	5.71	144.60	6.08	182.30	6.45	225.80	6.80	263.70	6.85	317.10	7.17	376.40	7.48
0.56	79.70	5.01	110.10	5.61	141.90	5.97	179.00	6.33	221.60	6.68	258.80	6.72	311.20	7.04	369.40	7.35
0.54	78.30	4.92	108.20	5.51	139.40	5.87	175.90	6.22	217.60	6.56	254.30	6.61	305.80	6.92	363.00	7.22
0.52	76.80	4.83	106.10	5.40	136.80	5.76	172.50	6.10	213.60	6.44	249.50	6.48	300.00	6.78	356.10	7.08
0.50	75.30	4.74	104.10	5.30	134.10	5.64	169.20	5.98	209.50	6.31	244.60	6.36	294.20	6.65	349.20	6.94
0.48	73.80	4.64	102.00	5.19	131.50	5.53	165.80	5.87	205.30	6.19	239.60	6.23	288.40	6.52	342.30	6.81
0.46	72.20	4.54	99.80	5.08	128.60	5.41	162.20	5.74	200.90	6.05	234.60	6.10	282.10	6.38	334.90	6.66
0.44	70.60	4.44	97.60	4.97	125.80	5.29	158.70	5.61	196.40	5.92	229.40	5.96	275.90	6.24	327.50	6.51
0.42	69.00	4.34	95.40	4.86	122.90	5.17	155.10	5.48	192.00	5.78	224.20	5.83	269.60	6.10	320.00	6.36
0.40	67.30	4.23	93.00	4.74	119.90	5.04	151.20	5.35	187.30	5.64	218.60	5.68	263.00	5.95	312.10	6.21
0.35	63.00	3.96	87.10	4.44	112.30	4.73	141.70	5.01	175.40	5.28	204.80	5.32	246.30	5.57	292.40	5.81
0.30	58.40	3.67	80.70	4.11	103.90	4.33	131.10	4.64	162.40	4.89	189.60	4.93	228.00	5.16	270.70	5.38
0.25	53.30	3.35	73.60	3.75	94.90	[illegible]	119.70	4.23	148.20	4.43	173.00	4.50	208.10	4.71	247.00	4.91
0.20	47.60	2.99	65.80	3.35	84.60	3.57	107.00	3.78	132.40	3.99	154.70	4.02	186.00	4.21	220.60	4.39

Values of 'n'

Selection of the proper value of the coefficient of friction 'n' to be used in the hydraulic computations for sewers is most important. A great deal of study has been done in recent years on the value of 'n'. Among those who have carried out such studies is Richard D. Pomeroy who presented a paper on the subject at the 38th Annual Conference of the Water Pollution Control Federation in Atlantic City in October 1965. Dr. Pomeroy concluded that the asbestos cement sewers tested showed better coefficients than the vitrified clay sewers; the concrete sewers were poorer. He also concluded that poor construction, including irregularities of slope is often a cause of poor overall coefficients. The average values found from his test were 0.0122 for asbestos cement, 0.0136 for vitrified clay and 0.0165 for concrete. However, two of the concrete lines tested were abnormally fouled with gravel or sand and if these lines are discarded the average value of 'n' for concrete pipe was 0.0156. He did warn, however, that caution should be used before drawing conclusions about concrete pipe in general.

The Ontario Concrete Pipe Association has been quite active in the past couple of years in promoting a lower value for 'n' than that previously used for concrete pipe. "Design Data 14" published by the American Concrete Pipe Association deals quite extensively with the Manning 'n' values and the History of Research.

Again the key seems to be the type of joint used. Tests were made on pipes installed in two ways (1) with the pipe laid comparable to normal construction practice closely simulating field measurements of joint irregularities and (2) with pipe laid with extreme care to eliminate, as far as possible, all flow interference at the joints. The first condition was referred to as "average" joints and the second as "good" joints. Numerical values found from the testing of 36-inch and 24-inch tamped pipe with average joints were 0.0111 and 0.0110 respectively.

The O.C.P.A. issued a Bulletin in April 1971 wherein they report the results of a Survey of Ontario Municipalities relating to their design practice for sanitary and storm sewers. This Bulletin is reprinted on Pages 17 to 25 inclusive.

You may draw your own conclusions from this survey. The values of 'n' as recommended by W.P.C.F. for those type of pipe material we most commonly use for sewers are,

Asbestos Cement Pipes	'n' = 0.011 - 0.015
Concrete Pipe	'n' = 0.011 - 0.015
Corrugated Metal Pipe (½ x 2 2/3 in. corrugations)	
Plain	'n' = 0.022 - 0.026
Paved Invert	'n' = 0.018 - 0.022
Spun asphalt lined	'n' = 0.011 - 0.015
Vitrified Clay Pipe	'n' = 0.011 - 0.015

It is also important to note that W.P.C.F. conclude that the value of 'n' does not depend on the size of the pipe.

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ONTARIO CONCRETE PIPE ASSOCIATION • SUITE 2 • 4891 DUNDAS ST. W. • ISLINGTON • ONTARIO • AREA CODE 416 TELEPHONE 231-3118

BULLETIN

71-1

April 19, 1971

The Ontario Concrete Pipe Association Technical Committee has just completed on Survey of Ontario Municipalities relating to their design practice for Sanitary and Storm Sewers.

It was found that Mannings "n" values ranged from .010 to .015 with .013 being most commonly used for smooth wall pipe. I would remind you that .013 is used by the Ontario Water Resources Commission Sanitary Engineering Division.

As you know, the Concrete Pipe Industry in Ontario has made significant improvements in their production techniques resulting in a high quality product designed to the tolerances for gasket joints.

In view of these observations and the desirability of adopting a province wide standard, it is the recommendation of the Ontario Concrete Pipe Association that a Mannings "n" value of .013 be used throughout Ontario for all sizes of smooth wall pipe for Sanitary Sewers.

W.J.Laari, P.Eng.
Director of Engineering

WJL/mp

DESIGN PRACTICE SURVEY

MANNING'S "N" SANITARY SEWERS

OCPA
FEB. 1971

29 REPLIES

CONCRETE

25 REPORT USE OF PRODUCT

2 USE .015
4 USE .015 UP TO SIZES RANGING FROM 18" TO 24"
.013 FOR LARGER
18 USE .013 AS STANDARD
1 USES .013 BUT ONLY > 18"

ASBESTOS CEMENT

26 REPORT USE OF PRODUCT

1 USES .015
11 USE .013
3 USE .012
6 USE .011
5 USE .010

VITRIFIED CLAY

21 REPORT USE OF PRODUCT

3 USE .015 UP TO SIZES RANGING FROM 18" TO 24"
.015 FOR LARGER
1 USES .013 BUT ONLY SIZES < 18"
12 USE .013
1 USES .012
2 USE .011
2 USE .010

PLASTIC

5 REPORT USE OF PRODUCT

2 USE .013
1 USES .011
2 USE .010

CMP

4 REPORT USE OF PRODUCT

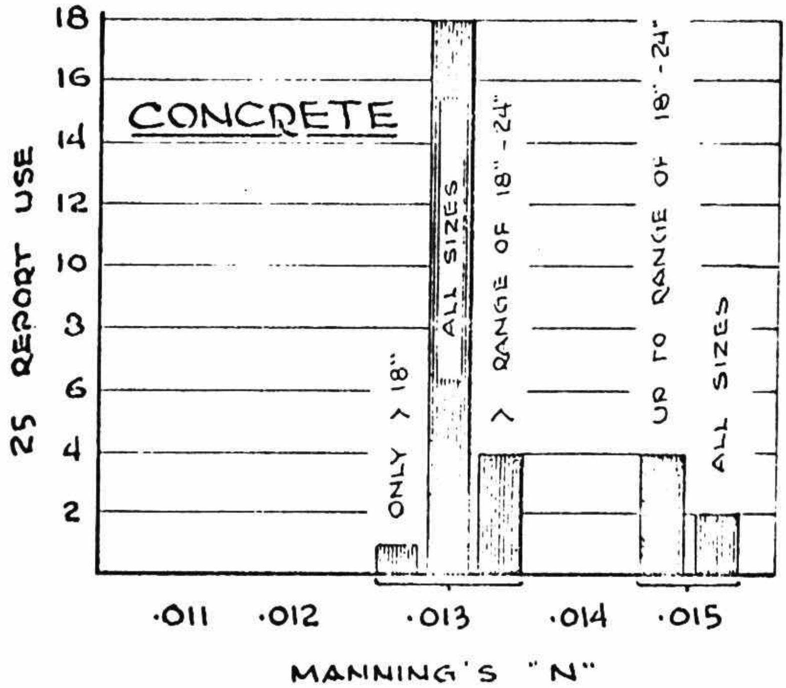
1 USES .027
3 USE .024

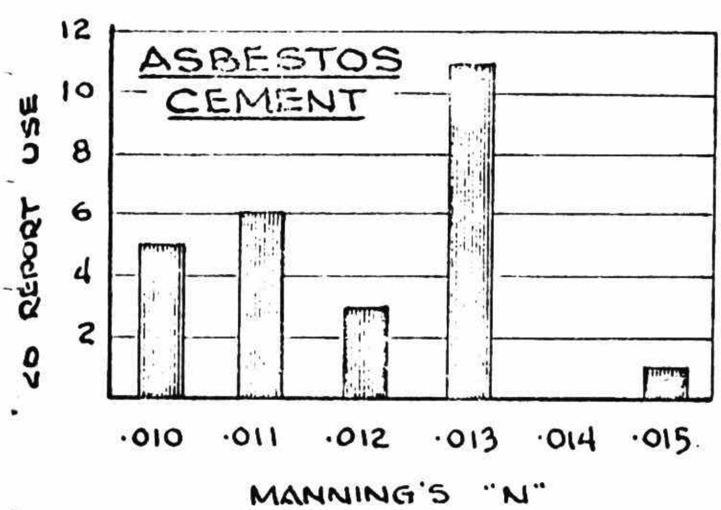
DESIGN PRACTICE SURVEY

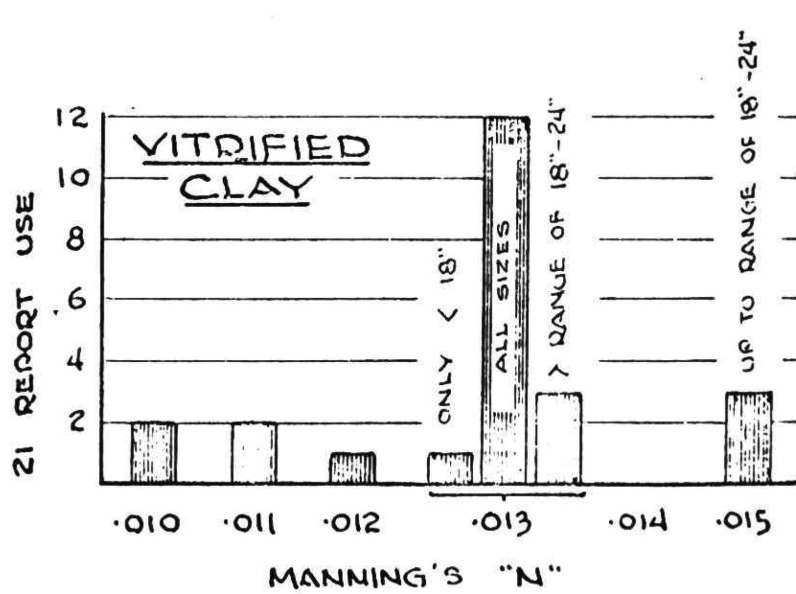
MANNING'S "N" SANITARY SEWERS

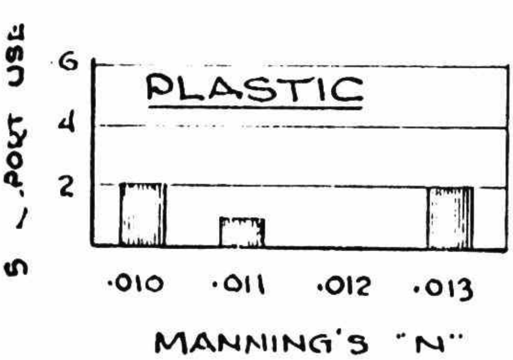
29 REPLIES

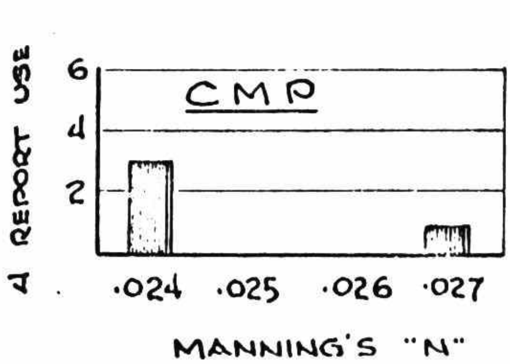
OCPA
FEB. 1971











DESIGN PRACTICE SURVEY

MANNING'S "N" SANITARY SEWERS

OCPA
FEB. 1971

	CONCRETE	VITRIFIED CLAY	ASBESTOS CEMENT	PLASTIC	CMP
ARRIE			.013		
ELLEVILLE	.013		.013	.010	.024
RAMPTON	.013	.013	.013		
URLINGTON	.013	.011	.010		
HATHAM	.013		.013		
HINGUACOUSY	.013	.013	.011		
ORNWALL			.012		
AST YORK	.015 TO 24" .013 > 24"	.015 TO 24" .013 > 24"			
TOBICOKE	.015	.015 TO 18" .013 > 18"	.011		
UELPH	.013	.012	.012		
ITCHENER	.013	.013	.013		.024
ONDON	.015 TO 18" .013 > 18"	.015 TO 18" .013 > 18"	.013		
ARKHAM	.013	.013	.011		
ISSISSAUGA	.013	.013	.013		
IAGARA FALLS	.015 TO 24" .013 > 24"	.010	.010		
ORTH BAY	.013		.010		
ORTH YORK	.015 TO 21" .013 > 21"	.013	.012		
TTAWA	.013	.013	.013		
WRC	.013	.013	.013		
EG. MUN. OF NIAGARA	.013		.010		
T. CATHARINES		.011	.011		
ARNIA	.015		.015		
CARBOROUGH	.013	.013	.011		
UDBURY			.013		
ORONTO (CITY)	.013	.013	.011	.010	
ORONTO (METRO)	.013	.013	.013	.013	.027
AUGHAN	.013	.010	.010		
ATERLOO	.013	.013	.011		

DESIGN PRACTICE SURVEY
SANITARY SEWERS

OCPA
FEB. 1971

	TYPE OF JOINT: TASK	TYPE OF JOINT: MOOT	TYPE OF JOINT: OTHER	ALLOW. INFILT^{N} & HEAD	ALLOW. EXFILT^{N} & HEAD	MATERIAL FOR "B" BEDDING
BARRIE	✓			INCREASE DESIGN FLOW BY 10%		DHO CLASS B
BELLEVILLE	✓			.5 G/IN.DIA/100'/HR		3/4" STONE
BRAMPTON	✓			.6 G/IN.DIA/100'/HR		3/4" CRUSHED STONE
BURLINGTON	✓		MECH. JT. ON AP PIPE	.25 G/IN.DIA/100'/HR	.25 G/IN.DIA/100'/HR	PEA GRAVEL OR CRUSHED STONE
CHATHAM	✓			.4 G/IN.DIA/100'/HR	.25 G/IN.DIA/100'/HR	3/4 TO 3/8 CRUSHED STONE
CHINGUACOUSY	✓					
CORNWALL	✓			.16 G/IN.DIA/100'/HR	.16 G/IN.DIA/100'/HR	GRANULAR "A"
EAST YORK	✓					3/4" CRUSHED QU
ETOBICOKE	✓			.6 G/IN.DIA/100'/HR DECREASES AS SIZE INCREASES OVER 36"	.4 G/IN.DIA/100'/HR 2' TO 15'	3/4 TO 3/8 CLEAR CRUSHED STONE
GUELPH	✓			.25 G/IN.DIA/100'/HR 2' TO 15'	.25 G/IN.DIA/100'/HR 2' TO 15'	1/2" GRAVEL OR 3/4" STONE
KITCHENER	✓		✓	.25 G/IN.DIA/100'/HR		3/4" CRUSHED STONE
LONDON	✓			.25 G/IN.DIA/100'/HR	INCREASE INFILT^{N} BY 25%	3/4" BLEND OF CRUSHED STONE
MARKHAM	✓					3/8" CRUSHED STONE
MISSISSAUGA	✓					3/4" CLEAR CRUSHED STONE
NIAGARA FALLS	✓			.25 G/IN.DIA/100'/HR	INCREASE INFILT^{N} BY 25% 2' TO 15'	
NORTH BAY	✓			.4 G/IN.DIA/100'/HR	.4 G/IN.DIA/100'/HR	3/4" CRUSHED STONE
NORTH YORK	✓			.25 G/IN.DIA/100'/HR	INCREASE INFILT^{N} BY 25%	3/4" CRUSHED STONE
OTTAWA	✓			.45 G/IN.DIA/100'/HR 25'	.45 G/IN.DIA/100'/HR 25'	3/8" STONE
OWRC	✓			.25 G/IN.DIA/100'/HR	INCREASE INFILT^{N} BY 25% 2' TO 15'	
REG. MUN. OF NIAGARA	✓					GRANULAR OR CONCRETE CRAD
ST. CATHARINES	✓		COUP^{L}			GRANULAR A OR
SARNIA	✓			.4 G/IN.DIA/100'/HR 1'	.4 G/IN.DIA/100'/HR 1'	
SCARBOROUGH	✓			.16 G/IN.DIA/100'/HR MAX 25'	.16 G/IN.DIA/100'/HR MAX. 25'	1500 PSI CONC. FROM BOTTOM TO S. LINE
SUDBURY	✓			.3 G/IN.DIA/100'/HR	.3 G/IN.DIA/100'/HR	SAND
TORONTO (CITY)	✓		MECH COUP^{L}	.5 G/IN.DIA/100'/HR	.5 G/IN.DIA/100'/HR 2'	3/4 TO 1/4 GRAVEL TO SPRING LINE
TORONTO (METRO)	✓			NIL		CRUSHED STONE OR 3/4 CLEAR
VAUGHAN	✓			CONC. .25 G/IN.DIA/100'/HR V.C. .2A " " A.C. .08 " "		3/4 TO 1/4 CLEAR CRUSHED STONE
WATERLOO	✓		TAT. PANN (CUM)	.4 G/IN.DIA/100'/HR	.4 G/IN.DIA/100'/HR 5'	PIT RUN GRAVEL 2" MAX.
YORK	✓			.25 G/IN.DIA/100'/HR	.25 G/IN.DIA/100'/HR	

DESIGN PRACTICE SURVEY

MANNING'S "N" STORM SEWERS

OCPA
FEB. 1971

29 REPLIES

CONCRETE 29 REPORT USE OF PRODUCT

1	USES	·015	
6	USE	·015	UP TO SIZES RANGING FROM 18" TO 24"
		·013	FOR LARGER
21	USE	·013	AS STANDARD
1	USES	·013	FOR SIZES < 30"
		·011	FOR LARGER

ASBESTOS CEMENT 13 REPORT USE OF PRODUCT

6 USE ·013
3 USE ·011
4 USE ·010

VITRIFIED CLAY 7 REPORT USE OF PRODUCT

5 USE ·013
1 USES ·011
1 USES ·010

PLASTIC 2 REPORT USE OF PRODUCT

BOTH USE ·010

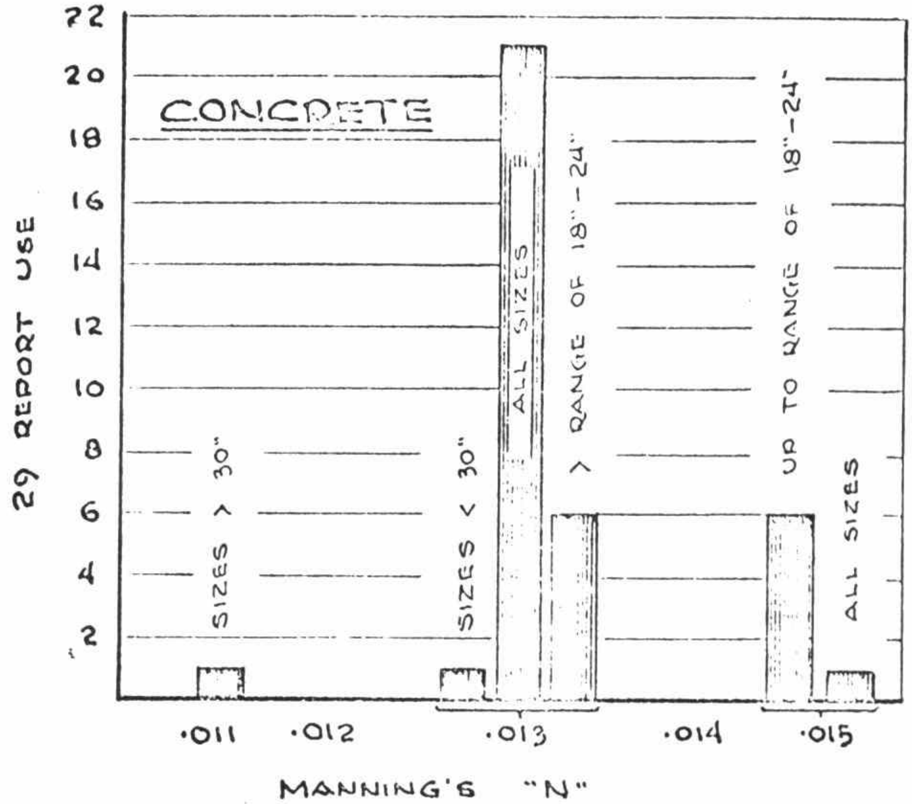
CMP 16 REPORT USE OF PRODUCT

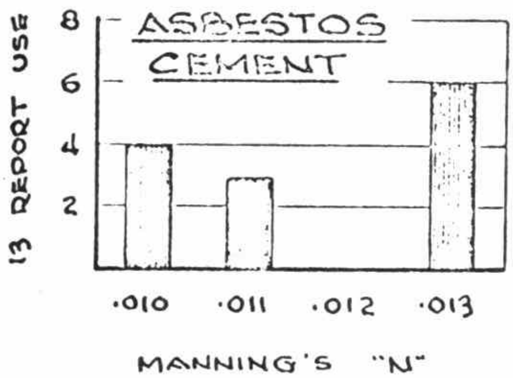
2 USE ·023
6 USE ·024
2 USE ·025
1 USES ·026
1 USES ·030
1 USES ·020 — ·025
1 USES ·021 — ·029
1 USES ·024 — ·027
1 USES ·025 — ·030

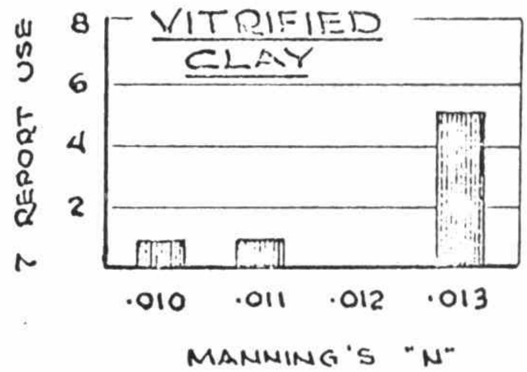
DESIGN PRACTICE SURVEY
MANNING'S "N" STORM SEWERS

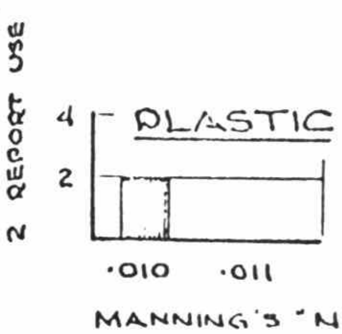
29 REPLIES

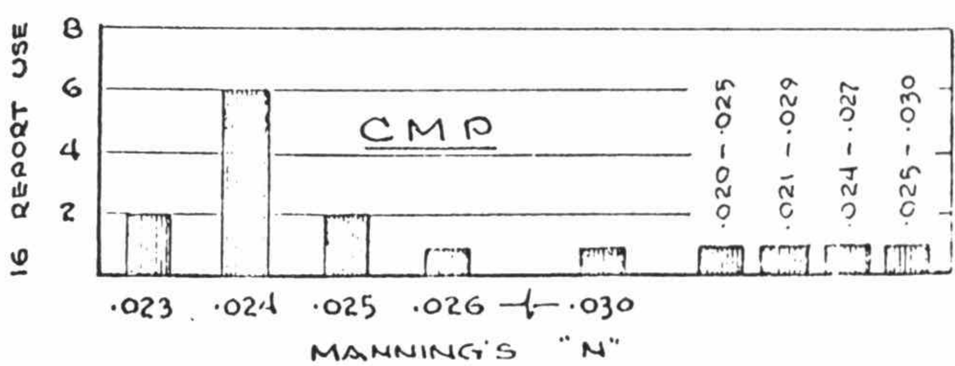
OCPA
FEB. 1971











DESIGN PRACTICE SURVEY

MANNING'S "N" STORM SEWERS

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	CONCRETE	VITRIFIED CLAY	ASBESTOS CEMENT	PLASTIC	CMP
ARRIE	·015 TO 24" ·013 > 24"				
ELLEVILLE	·013		·013	·010	·024
RAMPTON	·013				
URLINGTON	·013		·010		·025
HATHAM	·013		·013		
HINGUACOUSY	·013				
ORNWALL	·013				
AST YORK	·015 TO 24" ·013 > 24"				
TOBICOKE	·013				
UELPH	·013				·030
TCHENER	·013	·013	·013		·024
ONDON	·015 TO 18" ·013 > 18"				
ARKHAM	·013		·011		
ISSISSAUGA	·013	·013	·013		
AGARA FALLS	·015 TO 24" ·013 > 24"		·010		·021 TO ·02
ORTH BAY	·013		·010		·024
ORTH YORK	·015 TO 21" ·013 > 21"				2" CORR. ·0 1/2" CORR ·0
TTAWA	·013	·013	·013		·024
WRC	·013	·013	·013		·024
EG. MUN. OF NIAGARA	·013				·020 TO ·02
T CATHARINES	·013	·011	·011		·023
ARNIA	·015				·024
CARBOROUGH	·013				·024 TO ·027
UDBURY	·015 TO 21" ·013 > 21"	·013	·013		
ORONTO (CITY)	·013	·013	·011	·010	·023
ORONTO (METRO)	·013				·026
AUGHAN	·013	·010	·010		
ATERLOO	·013 TO 30" ·011 > 30"				·025
ORK	·013				

DESIGN PRACTICE SURVEY
STORM SEWERS

OCPA
FEB. 1971

	TYPE OF JOINT			ALLOW. INFILT^N & HEAD	ALLOW. EXFILT^N & HEAD	MATERIAL FOR "B" BEDDING
	GASK.	MORT.	OTHER			
BARRIE		✓		INCREASE DESIGN FLOW BY 10%		DHO CLASS
ELLEVILLE	✓			·5 G/IN.DIA/100'/HR		3/4" STONE
BRAMPTON	✓					3/4" CRUSHED STO
BURLINGTON	✓			·25 G/IN.DIA/100'/HR	·25 G/IN.DIA/100'/HR	PEA GRAVEL [illegible] CRUSHED STONE
CHATHAM		✓				DHO SPEC. OR CLASS B
CHINGUACOUSY	✓					3/4 TO 3/8 CRUSHED STONE
ORNWALL		✓		·16 G/IN.DIA/100'/HR	·16 G/IN.DIA/100'/HR	GRANULAR "A"
EAST YORK	✓					3/4" CRUSHER RU
ETOBICOKE	✓					3/4" TO 3/8" CLEAR CRUSHED STONE
GUELPH	✓			·5 G/IN.DIA/100'/HR	·5 G/IN.DIA/100'/HR	1/2" GRAVEL OR 3/4" CLEAR STONE
KITCHENER	✓	✓	✓			3/4" CRUSHED STO
ONDON	✓	✓				3/4" BLEND OF CRUSHED STON
MARKHAM	✓					3/8" CRUSHED STO
ISSISSAUGA	✓					3/4" CLEAR CRUSHED STON
NIAGARA FALLS	✓			·25 G/IN.DIA/100'/HR	INCREASE INFILT^N BY 25%	
ORTH BAY	✓	✓				3/4" CRUSHED STON
ORTH YORK				·25 G/IN.DIA/100'/HR	INCREASE INFILT^N BY 25%	3/4" CRUSHED STON
OTTAWA	✓	✓				3/8" STONE
UVRC						
REG. MUN. OF NIAGARA		✓				GRANULAR OR CONCRETE GRAVEL
T. CATHARINES		✓				GRANULAR "A" OR
SARNIA		✓		·4 G/IN.DIA/100'/HR	·4 G/IN.DIA/100'/HR	COMPACTED SA
SCARBOROUGH	✓			·45 G/IN.DIA/100'/HR	·45 G/IN.DIA/100'/HR	1500 PSI CONC. FR BOTTOM TO S.LIN
UDBURY	✓			·5 G/IN DIA/100'/HR	·5 G/IN.DIA/100'/HR	SAND
TORONTO (CITY)	✓		MECH. COUP.	·5 G/IN.DIA/100'/HR	·5 G/IN.DIA/100/HR 2'	3/4" TO 1/4 GRAVEL TO [illegible] LIN
ORONTO (METRO)	✓			NIL		CRUSHED [illegible] OR 3/4" CLEAR
VAUGHAN	✓			CONC. ·25 G/IN.DIA/100/HR V.C. ·28 " " " A.C. ·48 " " "		3/4" TO 1/4 CLEAR CRUSHED STONE
ATERLOO		✓		NO LIMIT	NO LIMIT	PIT RUN GRAVEL 2" MAX. SIZE
YORK	✓			·25 G/IN.DIA/100'/HR		

Hydraulic - elements graph

This graph as illustrated in Figure 24 on Page 27, is a most useful tool for calculating velocities and discharges for sewers flowing partially full. As can be seen on the graph there are curves for 'n' and 'f' which are variable with depth and 'n' and 'f' constant. In the paper referred to on Page 14 Richard Pomeroy concluded that velocities in partly filled pipes do not conform to the traditional equations thus the 'n', 'f' variable with depth curve.

W.P.C.F. caution that until more information and better analysis are available the decision to use a constant or variable 'n' must be left with the individual engineer.

I would like to go through an example illustrating how to use the hydraulics-element graph.

Given 24 inch diameter sewer laid at a grade of 0.20 percent. Find the velocity if the design flow is 7.- c.f.s. Assume Manning's 'n' = 0.013.

Solution From Chart on Page 8 Qf = 10.1 c.f.s.

Vf = 3.22 f.p.s. $\therefore \frac{Q}{Qf} = 0.69$

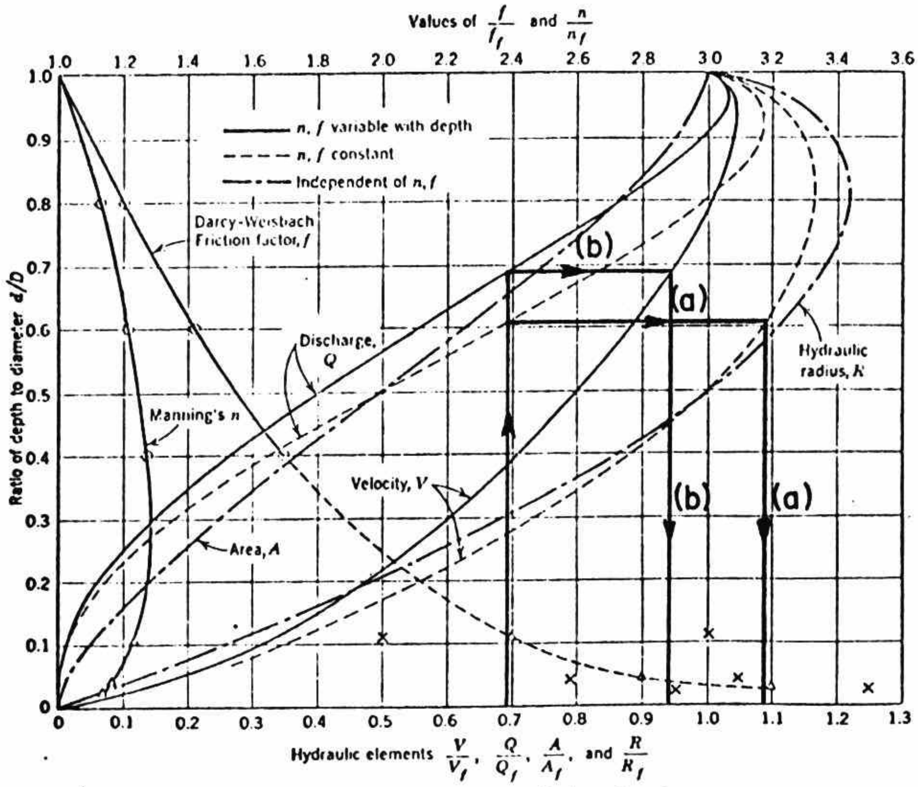


FIGURE 24.—Hydraulic-elements graph for circular sewers.

From Hydraulic-elements graph on Page 27

a) for 'n' constant $\frac{V}{Vf} = 1.09$

$\therefore V = 1.09 \times 3.22 = 3.51$ f.p.s.

b) for 'n' variable $\frac{V}{Vf} = 0.94$

$\therefore V = 0.94 \times 3.22 = 3.02$ f.p.s.

When designing sewers at flat grades with velocities at or near the minimum desired,care should be taken to check the actual velocity for low flow conditions. Low flow could occur during "non peak" hours or when the first few connections of an overall system are made. With a variable 'n' even worse conditions could be evident.

Minimum and Maximum Velocities

Minimum Velocities

The minimum desired velocity as common to most municipalities in Ontario is 2.5 f.p.s. when flowing full. However,sometimes it is necessary to design sewers at very flat grades and an acceptable minimum of 2.0 f.p.s. is used. W.P.C.F. prove that a velocity of 2.0 f.p.s. will transport organic particles 15.0 mm in diameter with a specific gravity of 1.01 and organic particles 0.74 mm in diameter with a specific gravity of 1.2 Sand 0.09 mm with a specific gravity of 2.65 will be transported effectively at 2.0 f.p.s.

Richard Pomeroy in his paper concludes that a velocity of 1.6 to 2.0 f.p.s. is needed to avoid excessive accumulations of debris.

In the design of sanitary sewers the velocity adequate for self cleansing should be obtained at the average or at least at the maximum flow at the beginning of the design period when the first connections have been installed. The sewer must also be adequately sized for the maximum flow at the end of the design period when all connections are made and design population reached.

Figure 25 on Page 30 illustrates the relationship between depth of flow and self cleansing velocities for circular sewers. No change in slope is required when the sewer flows more than half full. The slope would have to be doubled when depth of flow approaches 1/5 full and quadrupled when depth of flow reaches 1/10 full.

For storm sewers that sometimes are dry it is not possible to have scouring velocities at low flow. Self cleansing velocities should be attained for moderate storms.

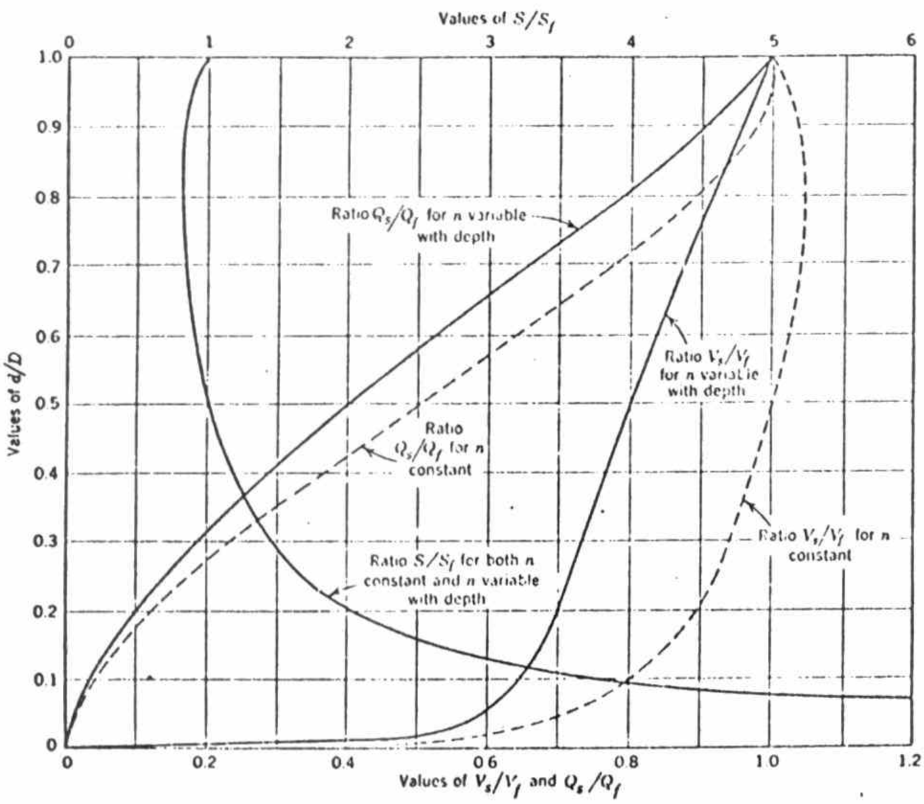


FIGURE 25.—Hydraulic elements of circular sewers that possess equal self-cleansing properties at all depths.

Maximum Velocities

Maximum velocities in sewers may be important because of the possibilities of excessive erosion on sewer inverts and because the liquid or sewage may "pile up" at the bottom end of a high velocity section of sewer due to a sudden reduction in velocity.

Maximum design velocities for clear liquid in concrete and vitrified clay pipe can be very high. Evidence is available that velocities in excess of 40 f.p.s. have been harmless to smooth, dense concrete and vitrified clay surfaces. Considerable erosion may be caused by gritty particles at much lower velocities. Care should be taken to reduce the velocity at the lower end of steep sections and at storm sewer outlets to prevent soil erosion.

A number of municipalities in Southern Ontario set a limiting velocity of 10 f.p.s. on sanitary sewers and 15 f.p.s. on storm sewers. Values of 12 to 15 f.p.s. for sanitary sewers and 20 to 22 f.p.s. for storm sewers could quite safely be used.

Factors to be Considered in the Design of Culverts

The discharge capacity of culverts is controlled by inlet or outlet conditions.

Inlet Control

In inlet control design the control section is located at or near the culvert entrance and for any given size and shape of culvert the discharge is dependent only on the inlet geometry and headwater depth. With inlet control the culvert flows part full. Slope, length and roughness coefficient do not affect the capacity of culverts governed by inlet control.

Outlet Control

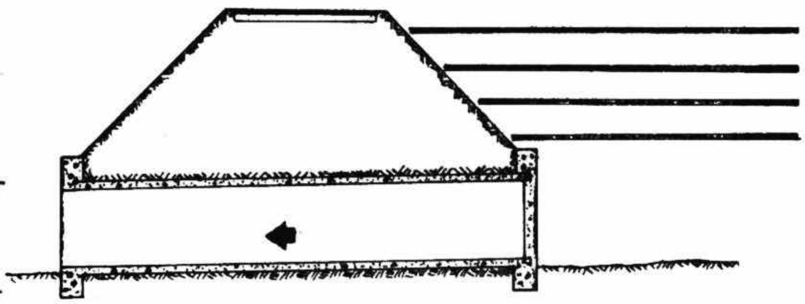
In outlet control design the control section is located at or near the culvert outlet. The discharge in this case is dependent on the shape, slope, length, roughness coefficient, tailwater depth, headwater depth and inlet geometry. Culverts operating under outlet control can flow either full or partially full.

Examples of relative headwater depths for various types of culverts are shown on the accompanying diagrams on Page 33.

CONTROL OF CULVERT FLOW

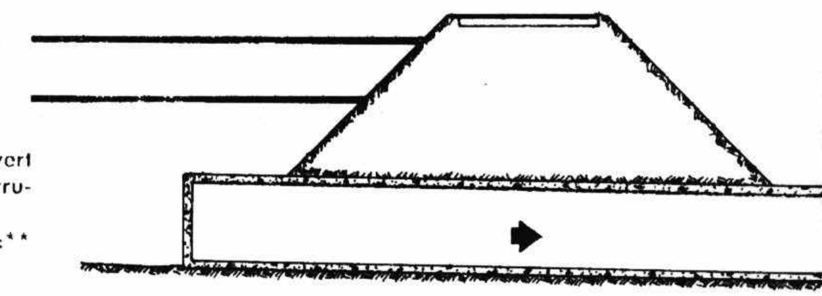
The point of control is located either at the inlet or the outlet. With inlet control, the culvert flows part full.

...s* with Inlet Control



With outlet control, the culvert usually flows full.

Relative Headwater Depths* ...



The reduced friction effect of smooth-wall pipe permits culvert flow at less headwater depth than for an equal size of corrugated-wall pipe.

Friction factors as reported by the University of Minnesota:**

Concrete pipe, $n = 0.010$
Corrugated pipe, $n = 0.025$

Friction is particularly important in long culverts. With modern highway design practices, most culverts can be considered as long culverts.

SUMMARY

Concrete pipe will always carry more water at the same headwater depths than corrugated metal pipe of the same size.

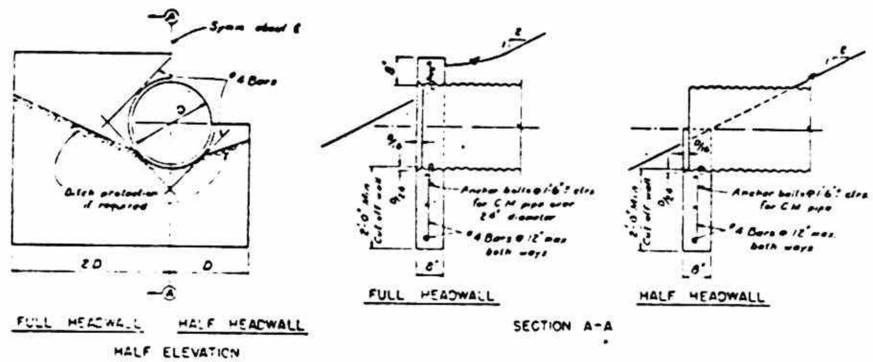
Comparison of culvert design should be made on the basis of hydraulic equivalency. To carry the same flow at comparable headwater depths, corrugated metal pipe must be at least one size, often two sizes, larger.

*Relative headwater depth as shown in the film for the same discharge.

**L. G. Straub and H. M. Morris, "Hydraulic Data Comparison of Concrete and Corrugated Metal Culvert Pipes," Technical Paper No. 3, Series B (1950), St. Anthony Falls Hydraulic Laboratory, University of Minnesota.

Circulars Nos. 5 and 10 by the U.S. Bureau of Public Roads are most commonly used in culvert design.

Care must be exercised by the Engineer in the design of inlet and outlet ends of culverts to prevent failure. A few recommended end treatments by the U. S. Bureau of Public Roads are included on Pages 35 to 38 inclusive. It is also very important to reduce the flow velocity at the ends of culverts as they are normally higher than the natural channel can accommodate resulting in erosion.



INLET STRUCTURES FOR CULVERT SIZES 18 IN. TO 36 IN. DIAMETER

SUMMARY OF QUANTITIES				
Culvert Dia inches	Full Headwall		Half Headwall	
	Concrete Cu Yd	Reinforcing Steel Lbs	Concrete Cu Yd	Reinforcing Steel Lbs
18	0.57	45	0.18	15
24	0.85	65	0.26	20
30	1.15	85	0.34	25
36	1.50	115	0.43	35

Note Quantities shown are for C M pipe culvert

GENERAL NOTES

Design Specifications AASHO Standard Specifications for Highway Bridges, 1961, with tentative revisions through 1964

Concrete All concrete shall be Class A(AE) using Type II (low alkali) Portland cement and having a minimum 28 day compressive strength f'_c = 3000 psi. The air entraining agent shall be approved by the engineer prior to acceptance for use. All exposed edges shall be chamfered ¾ inch unless otherwise noted

Reinforcing Steel Reinforcing steel shall be deformed bars of intermediate, hard or rail steel grade conforming to ASTM A-15 or A-16

Anchor Bolts Bolt and nut material shall conform to ASTM A 307. Bolts and nuts shall be galvanized after fabrication in accordance with ASTM A 153. Anchor bolts are not required for concrete pipe

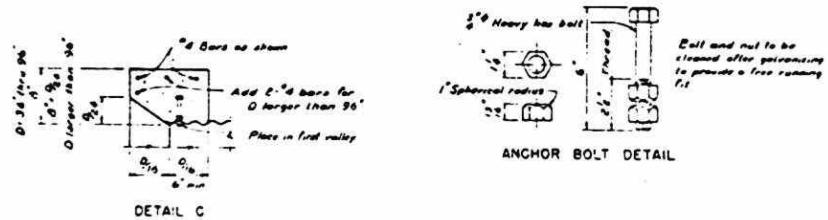
Cutoff Wall The depth of wall shown on the plan may be reduced if rock is encountered at a higher elevation

Multiple Pipe Installations To permit careful placing and tamping of back fill material, clear spacing for multiple pipe installations shall be not less than one-half the diameter of the larger pipe between sides of adjacent pipes but not required to exceed three feet, and in no case less than one foot

Piping When using pervious bedding and backfill, it is necessary to prevent seepage and piping by placing impervious material at the inlet. Cutoff collars may be used in lieu of impervious material

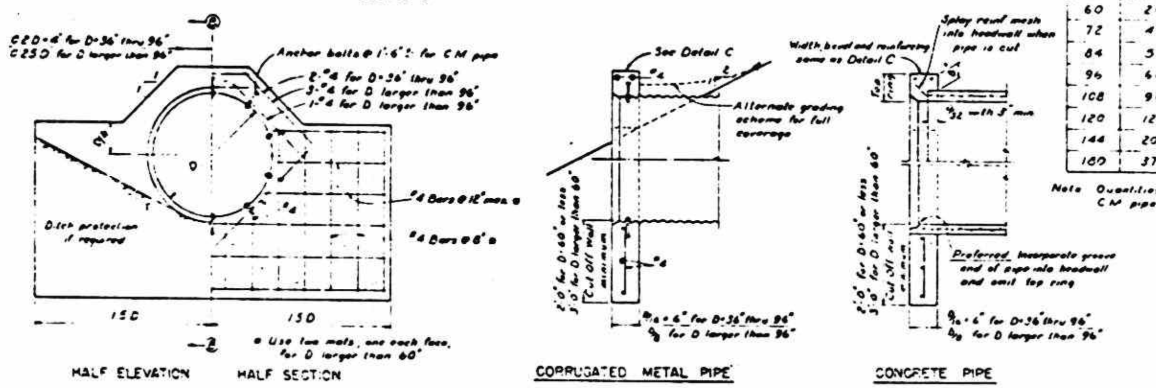
Skew When culvert is skewed to embankment, the embankment may be contoured as shown on Sheet No 4

Preformed End Sections Preformed end sections may be used in lieu of headwalls shown, if approved



SUMMARY OF QUANTITIES		
Culvert Dia inches	Concrete Cu Yd	Reinforcing Steel Lbs
36	10	75
42	13	90
48	15	105
54	20	125
60	24	145
72	41	455
84	56	535
96	68	640
108	98	795
120	124	955
144	201	1255
180	376	1820

Note Quantities shown are for C M pipe culvert



INLET STRUCTURE FOR CULVERT SIZES 36 IN. TO 180 IN. DIAMETER

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

CIRCULAR CULVERT END TREATMENT

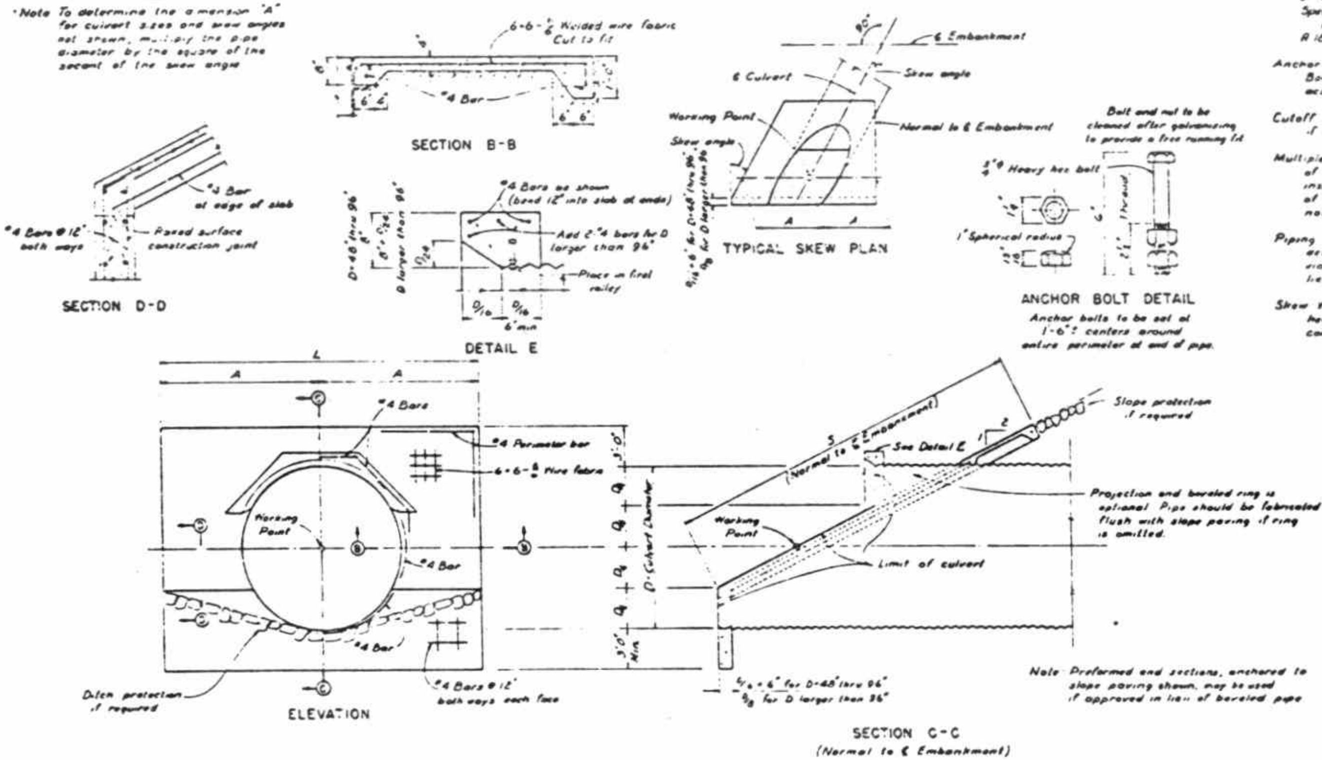
INLET STRUCTURES FOR CONCRETE AND CORRUGATED METAL CULVERTS SIZES 18 IN. TO 180 IN. DIAMETER

DO NOT SCALE	JULY 1964	SHEET NO.

TYPICAL DIMENSIONS AND QUANTITIES

Culvert Dia	S	Square Headwall				15° Skew				30° Skew				45° Skew				S	Culvert Dia
		A	L	Concrete	Reinf	A	L	Concrete	Reinf	A	L	Concrete	Reinf	A	L	Concrete	Reinf		
inches	ft-in	ft-in	ft-in	Cu Yd	Lbs	ft-in	ft-in	Cu Yd	Lbs	ft-in	ft-in	Cu Yd	Lbs	ft-in	ft-in	Cu Yd	Lbs	ft-in	inches
48	13-5	4-0	8-0	2.8	180	4-3	8-6	2.5	185	5-4	10-8	3.2	220	8-0	16-0	4.5	320	13-5	48
60	15-1⅛	5-0	10-0	3.7	225	5-4	10-8	3.7	235	6-8	13-4	4.4	290	10-0	20-0	6.3	420	15-1⅛	60
72	16-9¼	6-0	12-0	4.7	275	6-5	12-10	4.8	295	8-0	16-0	5.7	360	12-0	24-0	8.3	525	16-9¼	72
108	21-9⅝	9-0	18-0	8.9	480	9-8	19-4	9.3	510	12-0	24-0	11.3	630	18-0	36-0	16.8	930	21-9⅝	108
144	26-10	12-0	24-0	15.2	695	12-10	25-8	16.1	745	16-0	32-0	20.1	935	24-0	48-0	30.3	1395	26-10	144
180	31-10⅝	15-0	30-0	23.7	940	16-1	32-2	25.5	1020	20-0	40-0	32.1	1285	30-0	60-0	48.7	1945	31-10⅝	180

*Note To determine the dimension "A" for culvert sizes and skew angles not shown, multiply the pipe diameter by the square of the secant of the skew angle



GENERAL NOTES

Design Specifications AASHO Standard Specifications for Highway Bridges, 1961, with Interim revisions through 1964

Concrete All concrete shall be Class A(AE) using Type I (or air entr.) Portland Cement and having a minimum 28 day compressive strength f'_c = 3,000 psi. The air entraining agent shall be approved by the engineer prior to acceptance for use. All exposed edges shall be chamfered ¾ inch unless otherwise noted. Slope paving surface variations shall not exceed ⅛ inch in 10 feet

Reinforcing Steel Reinforcing steel shall be deformed bars of intermediate, hard or rail steel grade conforming to ASTM Specification A 15 or A 16. Welded wire fabric shall conform to ASTM Specification A 185

Anchor Bolts Bolt and nut material shall conform to ASTM A-307. Bolts and nuts shall be galvanized after fabrication in accordance with ASTM A-153

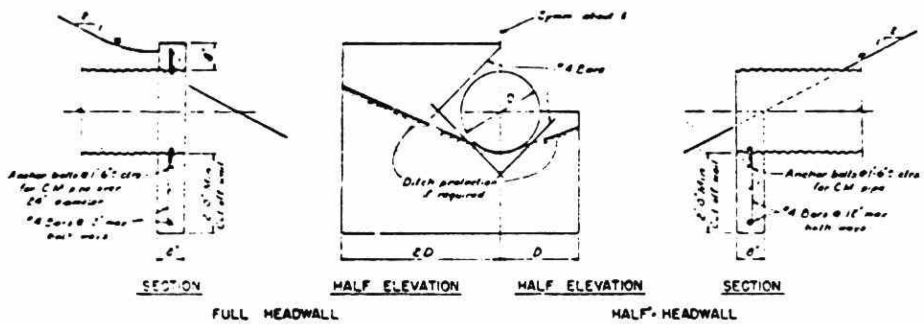
Cutoff Wall The depth of wall shown on the plan may be reduced if rock is encountered at a higher elevation

Multiple Pipe Installations To permit careful placing and tamping of backfill material, clear spacing for multiple pipe installations shall be not less than one half the diameter of the larger pipe between sides of adjacent pipes, but not required to exceed three feet

Piping When using pervious bedding and backfill, it is desirable to prevent seepage and piping by placing impervious material at the inlet. Cutoff covers may be used in lieu of impervious material

Skew When culvert is skewed to embankment slope, a square headwall may be used as an alternate and the embankment contoured as shown on Sheet No 4

U. S. DEPARTMENT OF COMMERCE BUREAU OF PUBLIC ROADS WASHINGTON, D. C.		
CIRCULAR CULVERT END TREATMENT		
INLET STRUCTURES FOR CORRUGATED METAL CULVERTS SIZES 48 IN. TO 180 IN. DIAMETER		
DO NOT SCALE	JULY 1966	SHEET NO. G-40-66



SECTION — HALF ELEVATION — HALF ELEVATION — SECTION

FULL HEADWALL — HALF HEADWALL

SUMMARY OF QUANTITIES				
Culvert Dia	Full Headwall		Half Headwall	
	Concrete	Reinforcing Steel	Concrete	Reinforcing Steel
inches	Cu Yd	Lbs	Cu Yd	Lbs
18	0.57	45	0.18	15
24	0.85	65	0.26	20
30	1.15	85	0.34	25
36	1.50	115	0.43	35

Note Quantities shown are for CM pipe culvert

GENERAL NOTES

Design Specifications AASHO Standard Specifications for Highway Bridges, 1961, with [illegible]

Concrete All concrete shall be Class A AE using Type I [illegible] Port land cement and having a minimum 28 day [illegible] strength f'c=3000 psi. [illegible] All exposed edges shall be chamfered ¾ inch unless otherwise noted

Reinforcing Steel Reinforcing steel shall be deformed bars of intermediate, hard or rail steel grade conforming to [illegible] A-15 or A-16

Anchor Bolts Bolt and nut material shall conform to [illegible] Bolts and nuts shall be galvanized after fabrication in accordance with ASTM A-153. Anchor bolts are not required for concrete pipe

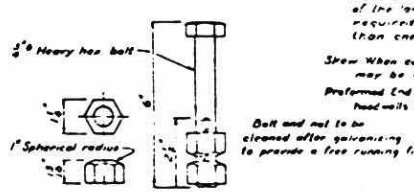
Cutoff Wall The depth of wall shown on the plan may be reduced if rock is encountered at a higher elevation

Multiple Pipe Installations To permit careful placing and tamping of back fill material, clear spacing for multiple pipe installations shall be not less than one half the diameter of the larger pipe between sides of adjacent pipes, but not required to exceed three feet, and in no case less than one foot

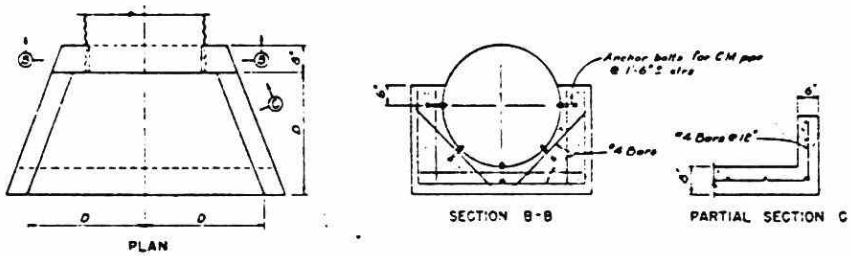
Skew When culvert is skewed to embankment slope the [illegible] may be contoured as shown on Sheet No. 4

Preformed End Sections Preformed end sections may be used in lieu of headwalls shown if approved

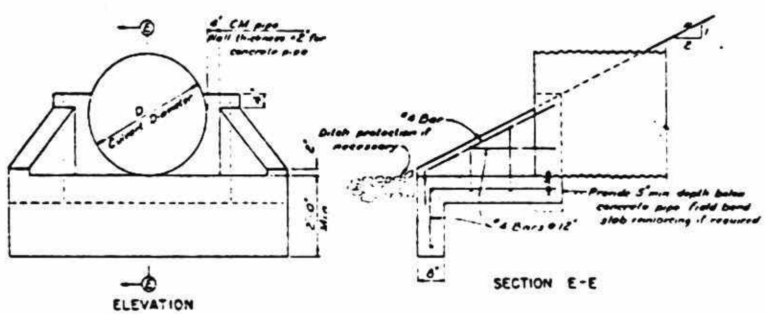
OUTLET STRUCTURE FOR CULVERT SIZES 18 IN. TO 36 IN. DIAMETER



ANCHOR BOLT DETAIL



PLAN — SECTION B-B — PARTIAL SECTION C



ELEVATION — SECTION E-E

SUMMARY OF QUANTITIES		
Culvert Dia	Concrete	Reinforcing Steel
inches	Cu Yd	Lbs
36	1.0	70
42	1.2	80
48	1.5	100
54	1.7	125
60	2.1	140

Note Quantities shown are for CM pipe culvert

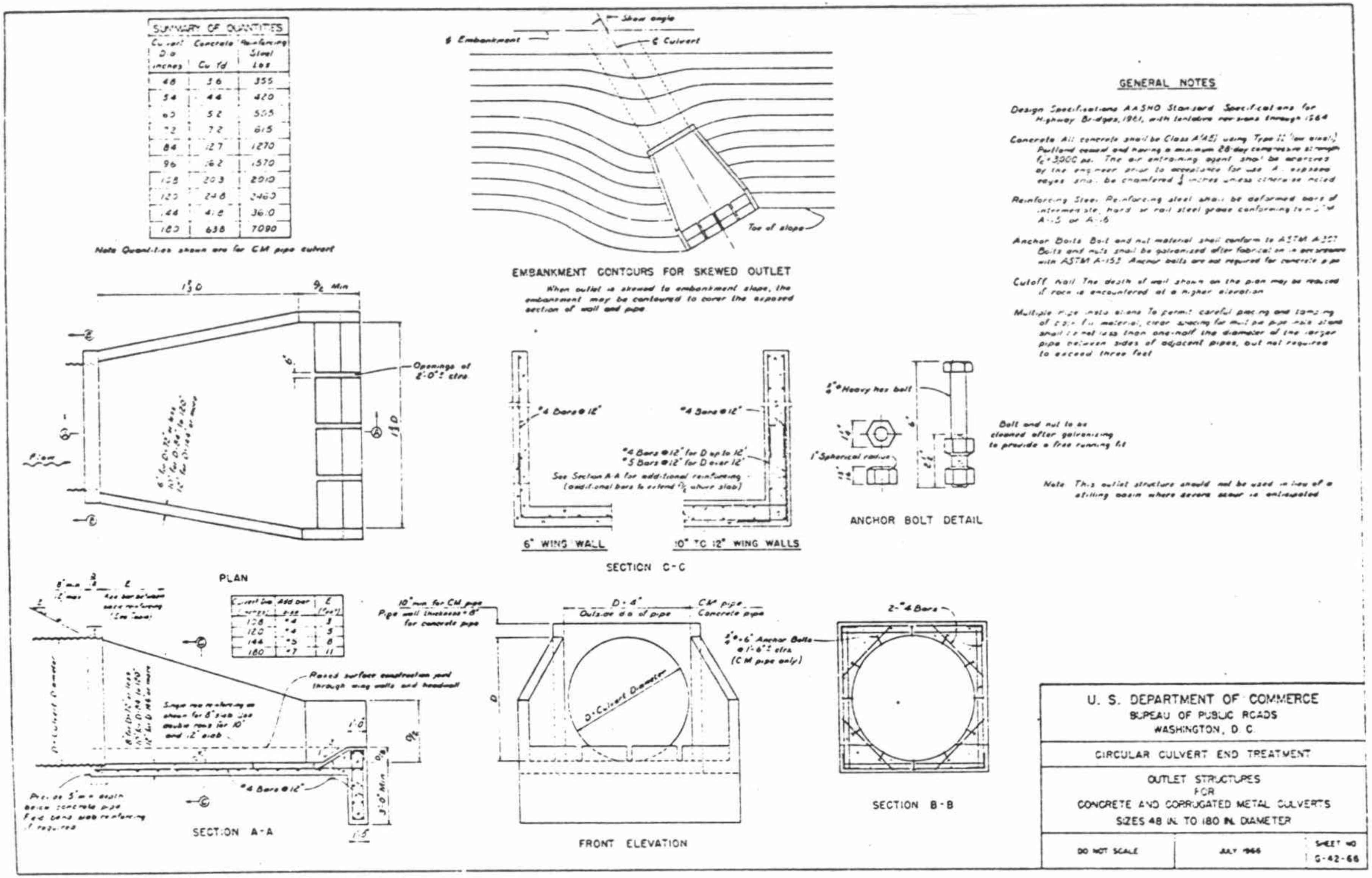
OUTLET STRUCTURE FOR CULVERT SIZES 36 IN. TO 60 IN. DIAMETER

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D. C.

CIRCULAR CULVERT END TREATMENT

OUTLET STRUCTURES
FOR
CONCRETE AND CORRUGATED METAL CULVERTS
SIZES 18 IN. TO 60 IN. DIAMETER

DO NOT SCALE	JULY 1965	SHEET NO. 1-4-56



SUMMARY OF QUANTITIES

Culvert Dia inches	Concrete Cu Yd	Reinforcing Steel Lbs
48	36	355
54	44	420
60	52	555
72	72	615
84	127	1270
96	162	1570
108	203	2010
120	248	2460
144	418	3610
180	638	7090

Note Quantities shown are for CM pipe culvert

Culvert Dia (inches)	Add bar size	E (feet)
108	"4	3
120	"4	5
144	"5	8
180	"7	11

GENERAL NOTES

Design Specifications AASHO Standard Specifications for Highway Bridges, 1961, with tentative revisions through 1964

Concrete All concrete shall be Class A (AE) using Type II (low alkali) Portland cement and having a minimum 28-day compressive strength f'_c = 3000 psi. The air entraining agent shall be approved by the engineer prior to acceptance for use. All exposed edges shall be chamfered ¾ inches unless otherwise noted

Reinforcing Steel Reinforcing steel shall be deformed bars of intermediate, hard or rail steel grade conforming to [illegible] A-15 or A-16

Anchor Bolts Bolt and nut material shall conform to ASTM A-307. Bolts and nuts shall be galvanized after fabrication in accordance with ASTM A-153. Anchor bolts are not required for concrete pipe

Cutoff Wall The depth of wall shown on the plan may be reduced if rock is encountered at a higher elevation

Multiple pipe installations To permit careful placing and tamping of back fill material, clear spacing for multiple pipe installations shall be not less than one-half the diameter of the larger pipe between sides of adjacent pipes, but not required to exceed three feet

Hydraulics of Sewer Junctions

A junction is said to occur when one or more branch sewers enter a main sewer. Manholes are the most common form of junctions.

The Borough of Scarborough has done considerable work in this area of sewer design. In 1969 they arranged for the University of Toronto to undertake research in the hydraulics of sewer junctions.

The results of this work can be found in the University of Toronto Library in a report entitled Hydraulics of Sewer Junctions by P. A. B. Gharghouri What they did was build an exact duplicate in model form of a sewer manhole that existed in the Borough of Scarborough. This particular manhole had been chosen as investigations had revealed that while water was backing up in the storm sewer junction manhole the downstream sewer was not flowing full.

The conclusions of the report state 1) "whenever possible, a model study should be carried out prior to the construction of a manhole junction, especially when the sewer pipes carry large flow rates", 2) "the occurence of manholes in which main and lateral flows are of some magnitude should be avoided", 3) "the angles of intersection should be as small as possible", 4) "when large angles are necessitated by local requirements it may be advisable to effect the major change in direction outside the chamber i.e. in the channel portion of the incoming flow, but to strive for gentle, well streamlined, junctions in the box itself."

Professor H. J. Leutheusser, P. Eng. of the University of Toronto states "the height of benching bears watching in the construction of manholes" and "indiscriminate improvements of design details which are not guided by consideration of the hydraulic performance of the structure as a whole may actually prove more harmful than no changes at all."

As a result of these studies the Borough of Scarborough have issued guidelines, design criteria and standard forms for hydraulic calculations for junction and transition manholes. This data is reprinted on Pages 41 to 45 for your information. An example calculation on Page 46 follows:

December 30, 1969

GUIDELINES FOR DETERMINATION OF SPACING OF MANHOLES FOR SEWER DESIGN IN THE BOROUGH OF SCARBOROUGH

Manholes are an essential element of any sewer system and generally are used to serve the following purposes:-

(1) access to sewers for inspection and maintenance,

(2) for most junctions with lateral sewers,

(3) for grade or alignment changes,

(4) for accommodating changes in pipe sizes.

With due consideration for the foregoing factors, the attached information has been prepared by a Committee represented by the Sewer Maintenance, Design, Construction, and Subdivision Control Sections, in an effort to set Guidelines for use in everyday sewer design in Scarborough.

It is a commonly accepted fact that manholes are costly and reduce the overall hydraulic efficiency of a sewer system. Therefore, in the preparation of these guidelines, attempts have been made to minimize the number of manholes. Particular emphasis has been given to reducing the number of junction manholes in storm sewers through the use of prefabricated Y fittings in storm sewers.

The information shown is intended for use on sewers in separate trenches as well as for sewers in the same trench.

It should be stressed that the information is to be used as a guideline only, and minor deviations will be allowed, provided they are approved by the Commissioner of Works.

SANITARY SEWERS: This information applies to all sizes used in Borough sanitary sewer design (10" - 27").

(a) Desirable manhole spacing .... 300 ft.
Maximum allowable spacing .... 400 ft.

(b) Manholes should be used at all junctions, changes in horizontal alignment, changes in grade, and changes in pipe size.

STORM SEWERS:

(a)

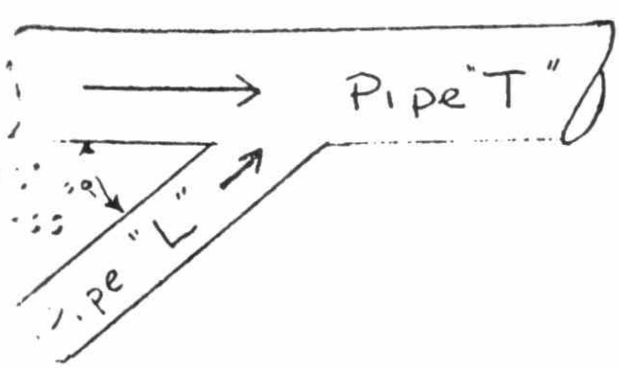
Diameter	Desirable Spacing	Maximum Allowable Spacing
12" to 30" incl.	300	400
33" to 48" incl.	400	500
54" to 72" incl.	500	700
Over 72"	700	800

(b) Manholes should be used at all changes in horizontal alignment in the 12" to 27" dia. range.

(c) Manholes should be used at all changes in grade; except for special designs involving use of elbows.

(d) Manholes should be used at all changes in pipe sizes; except for special designs in large size pipe.

(e) The use of prefabricated Y fittings is recommended for junctions in storm sewers under the following conditions:



(1) Pipe "T" MUST be 24" in diameter or larger.
(If pipe "T" is less than 24" dia. a manhole is required for the junction).

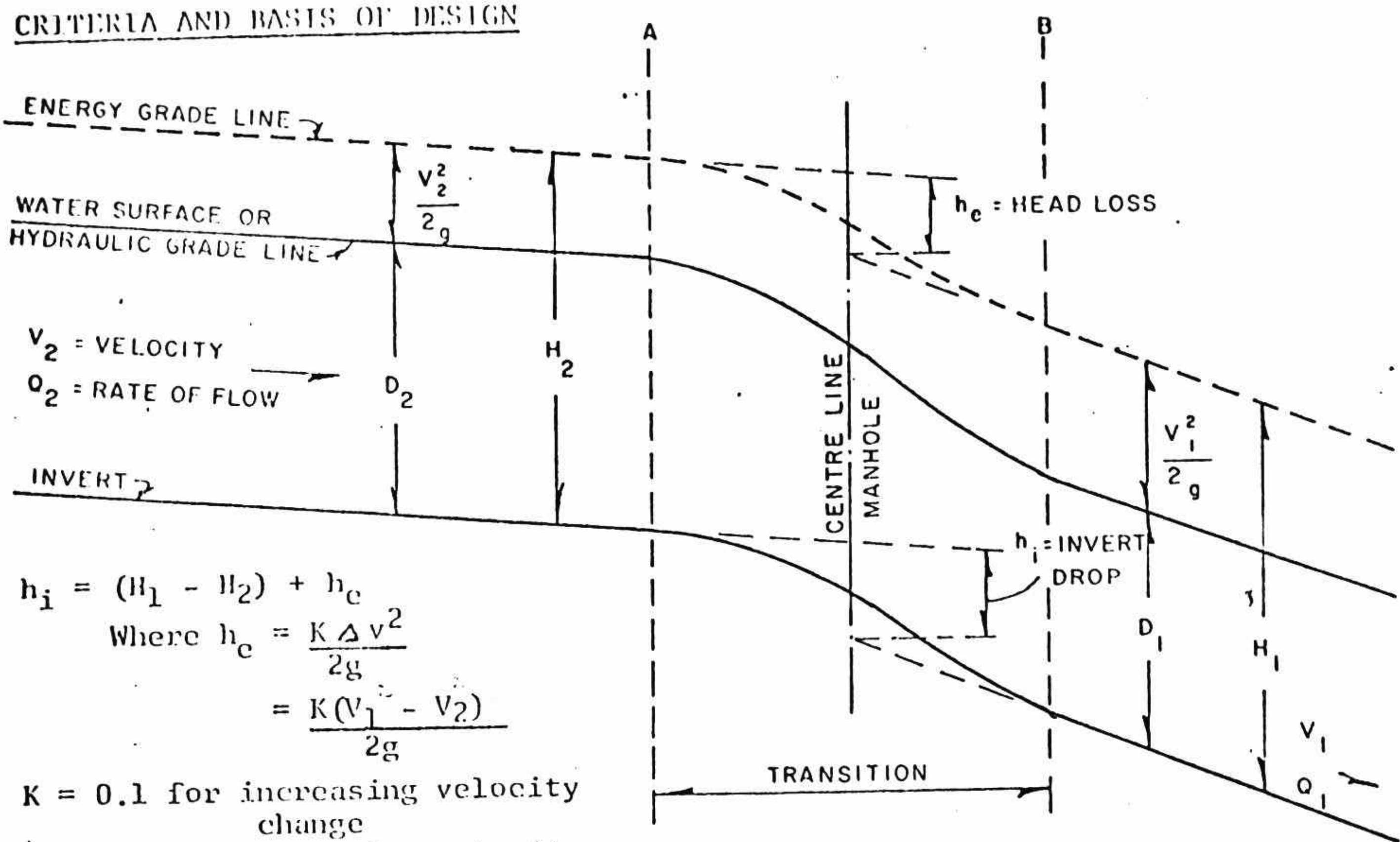
(2)

Diameter of Pipe "L"	Max. distance from point of junction to 1st M/H upstream on Pipe "L"
Less than 30"dia.	50 ft.
30" to 54" dia. incl.	400 ft.
Larger than 54"	600 ft.

HYDRAULIC CALCULATIONS

FOR JUNCTION AND TRANSITION MANHOLES

CRITERIA AND BASIS OF DESIGN



$$h_i = (H_1 - H_2) + h_c$$

Where $h_c = \dfrac{K \Delta v^2}{2g}$

$$= \frac{K(V_1 - V_2)}{2g}$$

K = 0.1 for increasing velocity change

K = 0.2 for decreasing velocity change

ASSUMPTION

Manhole length is relatively short so that h_i can effectively be taken to be the actual drop in inverts at the extremes of the manhole.

Each incoming pipe shall be analyzed separately together with the outgoing pipe.

REFERENCE

Employ Standard Drawing D-75 for % depth of flow and % velocity.

NOTE

The designer should, wherever possible, restrict the change in velocity to not more than 2.0 f.p.s. In special cases, consideration should be given to bellmouth entrances.

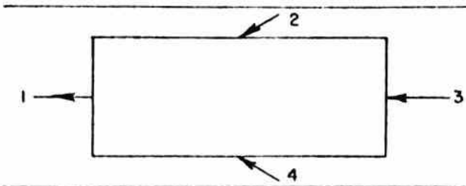
HYDRAULIC CALCULATIONS
For Junction and Transition Manholes

Location Manhole No. .... Designed by: ....

At .... Checked by: ....

.... Date: ....

Date ....



PIPE NO	DIAM. INCHES	GRADE %	CAPACITY Q cap (C.F.S.)	EXP FLOW Q act (C.F.S.)
1				
2				
3				
4				

Pipe No. 1 $Q_1 cap$ = ________ $Q_1 act$ = ________ $\frac{Q_1 act}{Q_1 cap}$ = ________

From D-75 read Depth of Flow = %

$V_1 cap$ = ________ from above depth of flow and D-75 read ratio of $V_1 act/V_1 cap$ = ________

$\therefore V_1 act = V_1 cap \times \frac{\_\_\_\_}{100}\% = \_\_\_\_ \times \frac{\_\_\_\_}{100} = \_\_\_\_$

H_1 = pipe diameter (feet) x % depth + $\frac{(V_1 act)^2}{2g}$ = ________ x ________ + ________ = ________

Pipe No. 2 $Q_2 cap$ = ________ $Q_2 act$ = ________ $\frac{Q_2 act}{Q_2 cap}$ = ________

From D-75 read Depth of Flow = %

$V_2 cap$ = ________ from above depth of flow and D-75 read ratio of $V_2 act/V_2 cap$ = ________

$\therefore V_2 act = V_2 cap \times \frac{\_\_\_\_}{100}\% = \_\_\_\_ \times \frac{\_\_\_\_}{100} = \_\_\_\_$

H_2 = pipe diameter (feet) x % depth + $\frac{(V_2 act)^2}{2g}$ = ________ x ________ + ________ = ________

Pipe No. 3 $Q_3 cap$ = ________ $Q_3 act$ = ________ $\frac{Q_3 act}{Q_3 cap}$ = ________

From D-75 read Depth of Flow = %

$V_3 cap$ = ________ from above depth of flow and D-75 read ratio of $V_3 act/V_3 cap$ = ________

$\therefore V_3 act = V_3 cap \times \frac{\_\_\_\_}{100}\% = \_\_\_\_ \times \frac{\_\_\_\_}{100} = \_\_\_\_$

H_3 = pipe diameter (feet) x % depth + $\frac{(V_3 act)^2}{2g}$ = ________ x ________ + ________ = ________

Pipe No. 4 $Q_4 cap$ = ________ $Q_4 act$ = ________ $\frac{Q_4 act}{Q_4 cap}$ = ________

From D-75 read Depth of Flow = ________ %

$V_4 cap$ = ________ from above depth of flow and D-75 read ratio of $V_4 act/V_4 cap$ = ________

$\therefore V_4 act = V_4 cap \times \frac{\_\_\_\_}{100}\% = \_\_\_\_ \times \frac{\_\_\_\_}{100} = \_\_\_\_$

H_4 = pipe diameter (feet) x % depth + $\frac{(V_4 act)^2}{2g}$ = ________ x ________ + ________ = ________

HEAD LOSS $h_e = K\frac{(V_1^2 - V_2^2)}{2g}$ for pipes 1 and 2

Select K = 0.1 or 0.2 as above

For pipes 1 and 2 h_e = ____________ (________ - ________) = ________

For pipes 1 and 3 h_e = ____________ (________ - ________) = ________

For pipes 1 and 4 h_e = ____________ (________ - ________) = ________

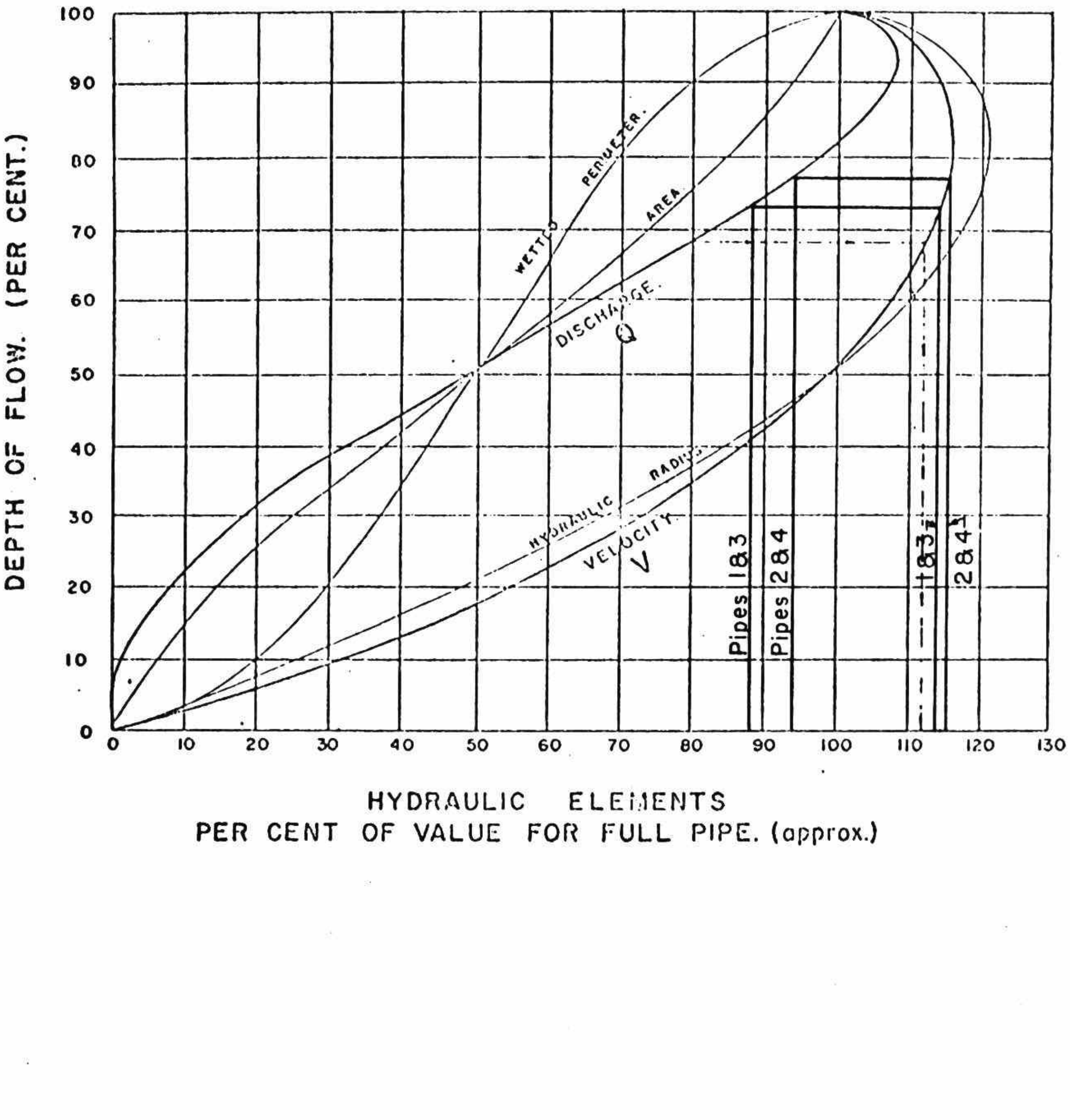
$\therefore$ for pipes 1 and 2 $h_i = (H_1 - H_2) + h_e$

h_i = ________ - ________ + ________ = ________ ' drop

for pipes 1 and 3 h_i = ________ - ________ + ________ = ________ ' drop

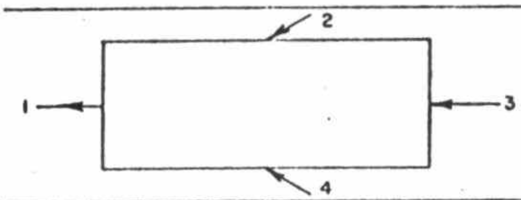
for pipes 1 and 4 h_i = ________ - ________ + ________ = ________ ' drop

SUMMARY Take maximum condition of the above three cases as the governing factor which sets the required maximum drop through the manhole.


DEPTH OF FLOW. (PER CENT.)
WETTED PERIMETER.
AREA.
DISCHARGE.
Q
HYDRAULIC RADIUS
VELOCITY.
V
Pipes 1&3
Pipes 2&4
1&3
2&4
0
10
20
30
40
50
60
70
80
90
100
110
120
130
HYDRAULIC ELEMENTS
PER CENT OF VALUE FOR FULL PIPE. (approx.)

HYDRAULIC CALCULATIONS
For Junction and Transition Manholes

Location Manhole No. .... Designed by: ....
At .... Checked by: ....
:: .... Date: ....
Date ....



PIPE NO.	DIAM. INCHES	GRADE %	CAPACITY Qcap (C F S)	EXP FLOW Qact (C F S)
1	30	0.80	36.70	32.00
2	15	1.00	6.47	6.00
3	24	1.00	22.60	20.00
4	15	1.00	6.47	6.00

Pipe No. 1 Q_1cap = 36.7 Q_1act = 32.0 $\frac{Q_1act}{Q_1cap}$ = 0.87

From D-75 read Depth of Flow = 73 %
V_1cap = 7.47 from above depth of flow and D-75 read ratio of V_1act/V_1cap = 113%
$\therefore V_1act = V_1cap \times \frac{113}{100}\% = 7.47 \times \frac{113}{100} = 8.45$

H_1 = pipe diameter (feet) x % depth + $\frac{(V_1act)^2}{2g}$ = 2.50 x .73 + $\frac{8.45^2}{64.4}$ = 2.93'

Pipe No. 2 Q_2cap = 6.47 Q_2act = 6.00 $\frac{Q_2act}{Q_2cap}$ = 0.93

From D-75 read Depth of Flow = 78 %
V_2cap = 5.27 from above depth of flow and D-75 read ratio of V_2act/V_2cap = 115%
$\therefore V_2act = V_2cap \times \frac{115}{100}\% = 5.27 \times \frac{115}{100} = 6.05$

H_2 = pipe diameter (feet) x % depth + $\frac{(V_2act)^2}{2g}$ = 1.25 x .78 + $\frac{6.05^2}{64.4}$ = 1.55'

Pipe No. 3 Q_3cap = 22.60 Q_3act = 20.00 $\frac{Q_3act}{Q_3cap}$ = 0.88

From D-75 read Depth of Flow = 73 %
V_3cap = 7.20 from above depth of flow and D-75 read ratio of V_3act/V_3cap = 113%
$\therefore V_3act = V_3cap \times \frac{113}{100}\% = 7.20 \times \frac{113}{100} = 8.14$

H_3 = pipe diameter (feet) x % depth + $\frac{(V_3act)^2}{2g}$ = 2.00 x .73 + $\frac{8.14^2}{64.4}$ = 2.49'

Pipe No. 4 Q_4cap = 6.47 Q_4act = 6.00 $\frac{Q_4act}{Q_4cap}$ = 0.93

From D-75 read Depth of Flow = 78 %
V_4cap = 5.27 from above depth of flow and D-75 read ratio of V_4act/V_4cap = 115%
$\therefore V_4act = V_4cap \times \frac{115}{100}\% = 5.27 \times \frac{115}{100} = 6.05$

H_4 = pipe diameter (feet) x % depth + $\frac{(V_4act)^2}{2g}$ = 1.25 x .78 + $\frac{6.05^2}{64.4}$ = 1.55'

HEAD LOSS $h_e = K\left(\frac{V_2^2 - V_1^2}{2g}\right)$ for pipes 1 and 2

Select K = 0.1 or 0.2 as above
For pipes 1 and 2 h_e = 0.1 $\left(\frac{6.05^2 - 8.45^2}{64.4}\right)$ = 0.05'

For pipes 1 and 3 h_e = 0.1 $\left(\frac{8.14^2 - 8.45^2}{64.4}\right)$ = 0.01'

For pipes 1 and 4 h_e = ______ (______ - ______) = 0.05'

$\therefore$ for pipes 1 and 2 $h_i = (H_1 - H_2) + h_e$
h_i = 2.93 - 1.55 + 0.05 = 1.43 ' drop

for pipes 1 and 3 h_i = 2.93 - 2.49 + 0.01 = 0.45 ' drop

for pipes 1 and 4 h_i = 2.93 - 1.55 + 0.05 = 1.43 ' drop

SUMMARY Take maximum condition of the above three cases as the governing factor which sets the required maximum drop through the manhole.

If invert of pipe 2 = 100.75

pipe 3 = 100.00

pipe 4 = 100.75

∴ required invert for pipe 1 is

(100.75 - 1.43) or (100.00 - 0.45) or (100.75 - 1.43)

= 99.32 or 99.55 or 99.32 = 99.32 which is the lowest.

Hydrogen Sulphide Generation and Related Problems

Certain specialized problems sometimes arise relative to sewer design. Our firm is involved in work internationally and has designed and supervised construction of sewers and treatment works in the Caribbean and the Middle East.

In hot climates, investigations must be made into the probability of hydrogen sulphide generation in the sewer system and steps that can be taken to minimize problems. On a recent sewer project in Jamaica despite high ambient temperatures, it was possible to minimize this problem in design of the system,since the topography permitted reasonable grades (and velocities) and the overall time of concentration was below a critical level.

In the preparation of Sanitary Drainage Master Plans and subsequent sewer designs for the Cities of Riyadh and Medinah in Saudi Arabia we had to contend with the likely generation of undesirable hydrogen sulphide gas within the sewer system. Local conditions contributing to the generation of this gas and its release are,high sewage temperatures due to the climate, sluggish and stagnant flow conditions due to flat topography and low flows during the initial life of the system.

Other conditions aggravating the situation include high sewage strengths, high sulphates in sewage and industrial waste, localized turbulent flow conditions that may be expected in manholes, and long flow times for the sewage in the system before treatment.

Hydrogen sulphide is an indicator of septicity in sewage and its emission should be avoided for several reasons.

- The gas in combination with moisture forms sulphuric acid which attacks structures containing Portland cement as well as many metals.
- The gas is toxic and can be dangerous to workers in sewers and manholes.
- The gas attacks paint on sewer structures and on neighbouring buildings.
- The odour of the gas is unpleasant and emerging from manholes can be a nuisance.

Nuisance conditions and damage to structure resulting from hydrogen sulphide can be reduced to a minimum by using sound design practices. Reasonable flow velocities with partially filled sewers will help to provide adequate and continuous ventilation by drawing air into the system and thereby preventing the release of gas. Non-perforated manhole covers will assist in containing gas within the system.

Where ventilation is inadequate due to low flow velocities the accumulated gas should be released above, rather than at street level. All house and building connections should, therefore, be adequately vented to the atmosphere at roof level. In addition, adequate gas traps should be provided in the internal sanitary plumbing of all buildings to prevent the release of gas within.

Location of Sewers

For the most part trunk and main sewers are located in valleys. In design of sewer systems it is important to consider future needs. Any sewer system or part thereof should be designed to take care of its present tributary area and still be compatible with the overall plan to serve the entire drainage area. Consideration should normally be given to building a future trunk sewer adjacent.

In most instances local sanitary sewers are located at or near the centreline of roads. Sometimes in built up areas it may be more economical to locate the sanitary sewer in a back of lot easement, or in a street boulevard.

Storm sewers are generally located within the roadway since their prime function is to receive roadway drainage and other surface flows via catchbasins or other storm water inlets.

Due consideration should be given to constructing sanitary and storm sewers in a common trench as savings can be achieved. In the case of local improvements on existing streets, sewers in common trench are almost mandatory in order to minimize disruption to the local residents and restoration costs.

Pipe Material

The most common materials used for sanitary sewers are vitrified clay, concrete and asbestos cement pipe.

The use of plastic pipe for sanitary sewers is receiving more and more consideration nowadays and quite often may be used for forcemains. Cast iron and ductile iron are commonly used for forcemains and river crossings.

In the case of storm sewers corrugated metal pipe can be used in addition to the materials used for sanitary sewers, and due to size limitations, asbestos cement and vitrified clay are less commonly used for storm sewers.

Sewer Depths

Where feasible,sewers should be deep enough to receive drainage by gravity. However,it may be more economical to provide individual pumping facilities for deep basements and buildings on land substantially below the street level.

In general,to provide gravity drainage, sewers should not be less than 3 feet below basement floors. A minimum of 4 to 5 feet of cover should be maintained to prevent freezing and uplift of sewer pipes. One must always check clearance of other utilities that might interfere with lateral connections and sewer crossings. For example a large water main of say 42 inches diameter with five feet of cover would effectively block connections to a sewer laid at the commonly used depth of 9 feet. The length of the house connection can also effect the sewer depth.

Infiltration and Exfiltration

Infiltration can cause harmful overloading of sewer systems and treatment plants. The selection of pipe joints and the method of connecting laterals are most important in minimizing infiltration.

Exfiltration can cause the washout of sewer bedding and settlement of the sewer resulting in caveins and structural failures in the sewers and pavements above.

The normal specification for infiltration allows in the order of 250 to 500 gpd/in.diam./mile. An infiltration test should be made where soil and ground water conditions indicate that the water table will be above the top of the sewer. Exfiltration tests are required where the water table will be below the invert of the sewer.

Modern equipment makes it quite easy to test sewers for exfiltration by such means as the air pressure test.

Computation Forms

Computations for determining required sewer sizes and slopes may be simplified by using forms similar to those attached hereto.

An example of how to use these forms will be illustrated for a typical street in a subdivision in the Metropolitan Toronto area. This example will also serve to show how economical pipe sizes are obtained and what minimum slopes should be used.

REFERENCES

1. W.P.C.F. Manual of Practice No. 9, 1969
"Design and Construction of Sanitary and Storm Sewers"

2. Calvin Victor Davis; Kenneth E. Sorensen;
"Handbook of Applied Hydraulics"

3. John W. Clark; Warren Viessman Jr.
"Water Supply and Pollution Control"

4. Richard D. Pomeroy, W.P.C.F. Journal, September 1967
"Flow Velocities in Small Sewers"

5. P.A.B. Gharghouri - University of Toronto Library
"Hydraulics of Sewer Junctions"

6. Ontario Concrete Pipe Association
"Bulletin 71-1"

7. American Concrete Pipe Association
"Design Data Manual"

8. The Corporation of the Borough of Scarborough
"Standard Drawings and Charts"

BOROUGH OF SCARBOROUGH

STORM SEWER DESIGN

DATE _______________

CALCULATED BY __________

CHECKED BY ____________

STREET ______________________

FROM ____________

TO ____________

ENTRY TIME - 10 MINUTES

M.H. TO M.H.		AREA				A.C.				TOTAL		Q	EXISTING OR PROPOSED SEWERS						
		RES.	SCHOOL	IND. or APT.	COMM.	·45	·60	·75	·90	A.C.	I	C.F.S.	LENGTH	SIZE	GRADE	CAP.	VEL.	TIME	TOT. TIME

JOB NO. _______

Q = C.I.A. cfs.
C = IMPERVIOUS COEFFICIENT
I = RAINFALL INTENSITY - ins./hr.
A = AREA - Acres

BOROUGH OF NORTH YORK

DEPARTMENT OF PUBLIC WORKS

STORM SEWER DESIGN

DESIGN SHEET NO. ___
ASSESS. SHEET NO. ___
SUBDIV. FILE NO. ___

STREET	MANHOLE		LENGTH	INCREMENT			TOTAL	FLOW TIME (min.)		I	TOTAL	S%	D	TYPE	Q	V	DESIGN FLOW		$d + \frac{v^2}{2g}$	M.H. INVERT
	FROM	TO	(ft.)	C	A	C.A.	C.A.	TO SECTION	IN SECTION		Q		(ins.)	of PIPE	FULL	FULL (fps.)	d (ft.)	v (fps.)	(ft.)	DROP (ft.)

JOB NO. ________ CALCULATED BY : ____________ CHECKED BY : ____________ DATE : ______

Q = AIR

R = COEFFICIENT OF RUN OFF

I = INTENSITY OF RAINFALL
FOR PERIOD IN INS./HR. N = 0.013

TOWN OF MARKHAM

STORM SEWER DESIGN

DESIGN SHEET NO. ____

DISTRICT OR
SUBDIV. NAME __________

2 YEAR CURVE ______________

LOCATION			AREA				AREA x COEFF.					I.	Q.	ELEVATION			LENGTH OF SEWER PIPE	SIZE PIPE	SLOPE		CAP. Q	VEL.	ENTRY TIME 5 MIN.	
STREET	MANHOLE										TOTAL	INS./HR.	C.F.S.	MH. FROM	MH. TO				PROVIDED				SECT. TIME	ACCUM. (MIN.)
	FROM	TO												INVERT	SURFACE	INVERT	(FT.)	(IN.)	(FT.)	%	(C.F.S.)	(F.P.S.)		

JOB NO. _______

CALCULATED BY: ____________ CHECKED BY: ____________ DATE: ________

- 215 -

Q = A. I. R.
R = COEFFICIENT OF RUN OFF.
I = INTENSITY OF RAINFALL
FOR PERIOD IN INS./HR.

STORM SEWER DESIGN

DESIGN SHEET NO. ____________
ASSESS SHEET NO. ____________
DRAINAGE AREA COLOUR ____________
SUB.- DIV. NAME ____________

LOCATION			RUN OFF						TOTAL		I	ACCUM. Q		ELEVATIONS			LENGTH OF SEWER PIPE	SIZE	SLOPE		ACTUAL		TIME (ENTRY 5 MIN.)	
STREET	MANHOLE		BACKYARD AREA			PAVED AREA								MH FROM	MH TO			PIPE	PROVIDED					
	FROM	TO	AREA AC.	R.	AR.	AREA AC.	R.	AR.	SECTION AR.	ACCUM. AR.		LATERAL C.F.S.	TRUNK C.F.S.	INVERT	SURFACE	INVERT		IN.	FT.	%	CAP. Q.C.F.S.	VEL. F.P.S.	SECT. MIN.	ACCUM. MIN.

CALCULATED BY: ____________ CHECKED BY: ____________ DATE: ____________

SUBDIVISION ____________________________

CONSULTANT ____________________________

DRAINAGE AREA PLAN Nº ________________

TOWN OF MISSISSAUGA

STORM SEWER DESIGN CHART

SHEET Nº ____________ OF ____________

PROJECT Nº ______________ n = __________

DESIGNED BY ____________ DATE __________

LOCATION	FROM M.H	TO M.H	AREA (acres)	RUNOFF COEFF	A x R	ACCUM. A x R	T OF C (minutes)	INTENSITY (in/hr)	EXPECTED FLOW (c.f.s.)	INVERT ELEVATION UPPER M.H	INVERT ELEVATION LOWER M.H	FALL	LENGTH	GRADIENT (ft/ft)	PIPE SIZE (inches)	CAPACITY (c.f.s.)	VELOCITY (f.p.s.)	TIME OF FLOW (minutes)	LOWER M.H

A-6221

SUBDIVISION ------------------

CONSULTANT ------------------

DRAINAGE AREA PLAN Nº ____________

TOWN OF MISSISSAUGA

SANITARY SEWER DESIGN CHART

SHEET Nº ---------- OF ----------

PROJECT Nº ---------- nº --------

DESIGNED -------- DATE ------

LOCATION	FROM M.H.	TO M.H.	AREA (acres)	DENSITY persons per acre	POPULATION	CUM AREA (acres)	CUM POP	SEWAGE * FLOW ① (c.f.s.)	INFILTRATION * FLOW ② (c.f.s.)	TOTAL FLOW ①+② (c.f.s.)	LENGTH (ft.)	PIPE DIA. (inches)	GRADIENT %	CAPACITY (c.f.s.)	VELOCITY (f.p.s.)	DROP IN LOWER M.H.

* REFER TO STD DWG A-7907. SEWAGE FLOWS FOR LIGHT INDUSTRIAL AND LIGHT COMMERCIAL SHALL BE BASED ON 30 ppa AND 20 ppa RESPECTIVELY

A-6220

LOCATION
from
to

TOWN OF BRAMPTON
Sewage Calculations and Design Sheet

CALCULATED BY:
CHECKED BY:
DATE

- STORM SEWAGE DESIGN -

Q = C I A (cfs)
RUNOFF CO-EFFICIENT (C) Parks = .25; Single & Semi-Detached = .50;
Multiples, Institutional, Industrial = .75; Commercial = .90
RAINFALL INTENSITY (I) 3.25" per hour. Inlet Time 10 Min.,
Time of Concentration 15 Min

- SANITARY SEWAGE DESIGN -

POPULATION METHOD: $Q = \frac{FDM}{540{,}000} \times A$
Average Daily Flow (F) = (100) gal/capita/day
Density (D) Persons per Acre
Ratio of Peak to Avg Flow (M) = 2.5

CO-EFFICIENT METHOD Q = C A
Co-Efficient (C): Residential = .015 cfs/Acre Apartments, 60 u.p. Acre = .050 cfs/Acre
Commercial - Industrial = .025 cfs/Acre High Density Redevelopment = .100 cfs/Acre
Schools & Mult.- Family = .035 cfs/Acre.

Se[illegible]	UPSTREAM			DOWNSTREAM			Nº OF ACRES			POPULATION			AREA x SANI CO-EFF					TOTAL SANI FLOW cfs	TIME	TOTAL TIME	AREA x STM CO-EFF				TOTAL AxC	I	TOTAL STM FLOW cfs	PIPE ROUGHNESS n =			
	MH	Gr'nd	Invert	MH	Gr'nd	Invert	In Area	Contrib	Total	Density	In Area	Total Acc Persons	.015	.025	.035	.050	.100				.25	.50	.75	.90				Length	Size	Grade	Ve[illegible]

- 219 -

Sheet Nº of

SANITARY SEWER DESIGN

FOR POPULATION < 1000 $M = q\left(1 + \frac{14}{4 + p^{0.5}}\right)$

FOR POPULATION > 1000 $M = \frac{5q}{p^{0.2}}$

POP. FLOW $Q = \frac{M \times P}{540}$

q = average daily flow in IGPCPD. (usually = 100)

M = peak daily flow in IGPCPD

$P = \frac{\text{Population}}{1000}$

Q = Population flow in C.F.S.

SHEET No ______ OF ______

JOB No ______________

SUBDIVISION ______________

____

LOCATION			SECTION		CUMULATIVE		M.	POP. FLOW	INFIL. 0.004	CUM. FLOW	ELEVATIONS			LENGTH OF SEWER	PIPE SIZE	TYPE OF PIPE	SLOPE		ACTUAL	
STREET	MANHOLE FROM	MANHOLE TO	POP.	AREA ACRES	POP.	AREA ACRES	IGPCPD	C.F.S.	C.F.S./AC.	C.F.S.	M.H. FROM INVERT	M.H. TO SURFACE	M.H. TO INVERT	FT.	IN.		FT.	%	CAP. C.F.S.	VEL. F.P.S.

D 56 A

CALCULATED BY ______________ DATE ______ CHECKED BY ______________ DATE ______

S T R U C T U R A L   A S P E C T S
O F   S E W E R   D E S I G N

STRUCTURAL DESIGN OF SEWERS

by

Mr. W. A. Elliott, P. Eng.

INTRODUCTION

In the design and selection of a pipe conduit to suit a particular situation, several factors must be considered. These are as follows:

1. Hydraulics - how large.
2. Economics - how expensive.
3. Structural Strength - how strong.
4. Durability - how long will it last.
5. Installation - how easy is it to construct.
6. Couplings and fittings - how watertight or how versatile in permitting connections to be made.

One can see that all six of these factors must be correlated and given some order of importance to fit the particular problem. It must be remembered that all of these factors are of equal importance in the final selection of a conduit. It would be wrong to install a pipe that is both adequate in size and strength but is of insufficient durability or is not watertight. Both cases could result in failure in a very short period of time.

It is the scope of this article to expound on the third factor - the structural aspects of sewer design.

Basically, there are two types of conduits available, - rigid, (concrete, vitrified clay or asbestos cement) and flexible, (steel, corrugated metal pipe, soft plastic). The performance of rigid and flexible pipes in supporting applied loads differs

considerably. The theory of loads and supporting strengths of each type of conduit is well documented and founded on tests and experiments dating over the past sixty years. Today, more sophisticated methods are being sought to analyze the soil-structure interaction. The Marston equations themselves are being questioned with the hope that a more accurate solution in predicting loads on buried structures will result.

The design procedure for the selection of rigid pipe involves the consideration of the following:

1. Determination of the earth load.
2. Determination of the live load, if applicable.
3. Selection of a field bedding condition.
4. Determination of a load factor.
5. Application of a factor of safety.
6. Determination of the required D-load.

1. Determination of the earth load

The theory of earth loads on buried conduits was derived by Martsonwho analyzed the problem using rational principles of mechanics. Actual life size model tests supported his theories.

Figure 1 categorizes the classifications of conduit installations. Each type will be briefly discussed later.

Figure 1
Classification of Underground Conduits

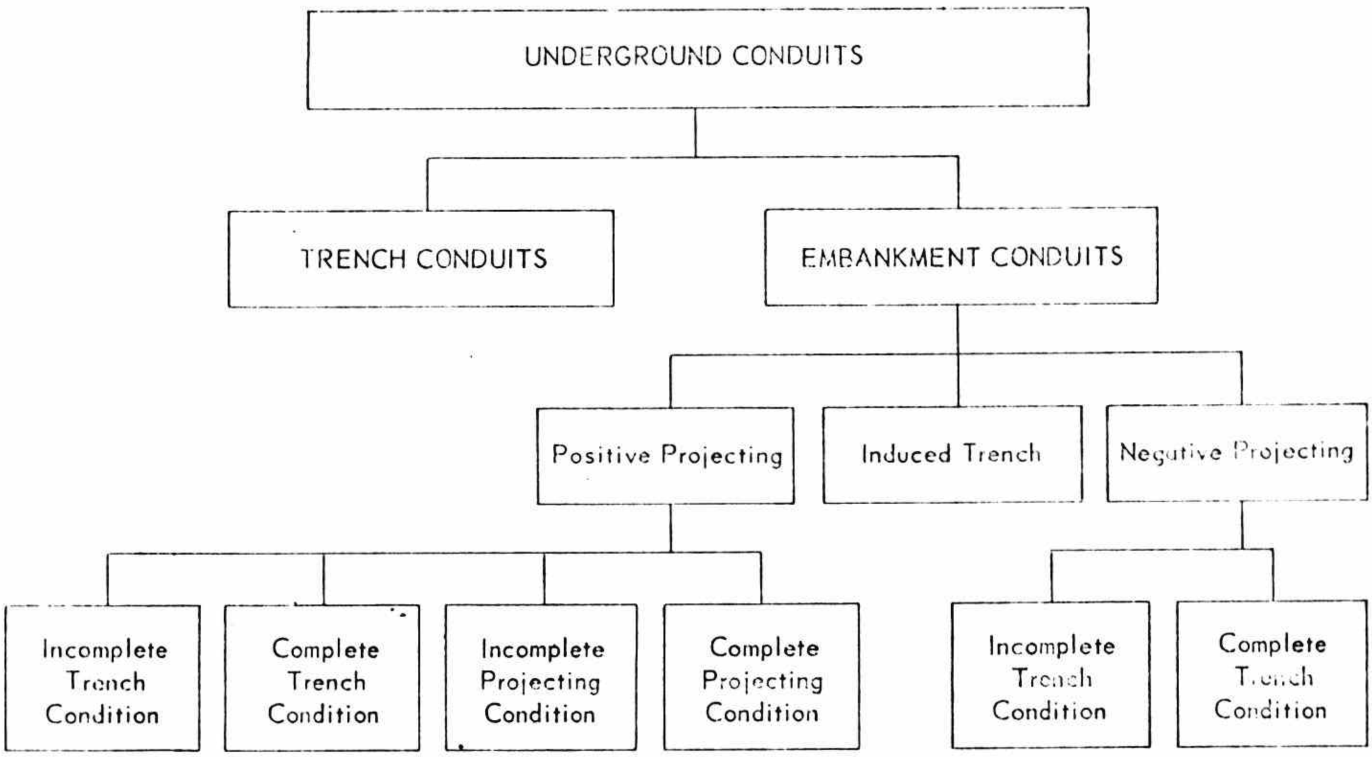
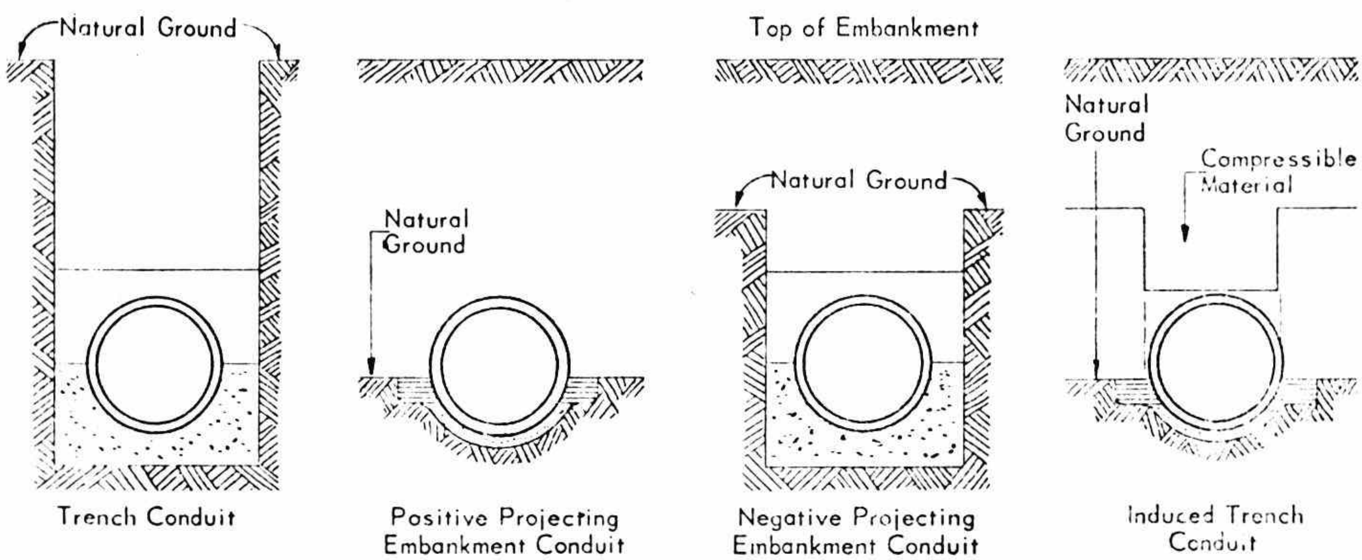


Figure 2
Essential Features of the Various Types of Installations



2. Determination of Live Loads

The distribution of stresses through a soil mass has long been established by Boussinesq and others and various charts and diagrams are now available to simplify the selection of a conduit.

3. Selection of a Field Bedding Condition

The selection of this factor is an arbitrary choice dependent upon the economics and degree of care of installation that the designer wishes to impose. The method of installation and control in enforcing that the design is followed is one of the prime problems encountered by the engineer. More failures arise due to lack of care in following the bedding specification than any other factor.

4. Determination of a Load Factor

This factor has been determined from experiment and is directly related to the care taken to maximize the amount of contact that the pipe barrel makes with the surrounding backfill material.

5. Application of a Factor of Safety

It is common in all engineering problems to design to the working stress which is the ultimate stress at failure divided by a factor of safety.

For rigid pipe design, an empirical factor of safety of 1.0 and 1.5 is recommended for reinforced and non-reinforced pipe respectively.

The structural strength of a rigid pipe is usually measured by a three-edge test. The strengths determined from these tests is then classified by D-loads. The D-load, an arbitrary parameter which measures the strength for the establishment of a standard, is the three-edge bearing strength divided by the internal diameter of the conduit. The following table lists the various D-loads for the more common pipe materials and specifications.

	Specification		Class	D-Load		
Plain Concrete	ASTM	C - 14 - 70	1	1800	p.l.f./ft. of dia.	**
		C - 14 - 70	2	2250	"	**
		C - 14 - 70	3	2600	"	**
Reinf. Concrete		C - 76 - 68	II	1000	"	
		C - 76 - 68	III	1350	"	
		C - 76 - 68	IV	2000	"	
		C - 76 - 68	V	3000	"	
Vitrified Clay	CSA	A 60.1	S.S.	1800 *	" for 12" dia.	
	CSA	A 60.1	E.S.	2600 *	" for 12" dia.	
Asbestos Cement	ASTM	C 428	1500	1500	"	
		C 428	2400	2400	"	
		C 428	3300	3300	"	
		C 428	4000	4000	"	
		C 428	5000	5000	"	

* not a linear relationship - does not vary directly with diameter.

TABLE 1

**tentative revision (not yet approved for incorporation in the ASTM standards C-14-70 or later edition).

The design of flexible pipes involves -

1. Determination of earth load.
2. Determination of live load, if applicable.
3. Investigation of seam strength.
4. Investigation of buckling strength.
5. Investigation of deflection.
6. Investigation of handling and ease of installation.
7. Application of a factor of safety to above

These steps shall be discussed more fully.

The intention of this article is to briefly outline the numerous methods of designing a conduit. Because of the lack of adequate time, only the highlights and personal observations can be touched upon. However, it is hopeful that this review of the structural aspects will whet the intellectual appetite of the novice or even the learned and that further interest will be generated to pursue this many-faceted and highly interesting topic.

FLEXIBLE PIPE DESIGN

The structural strength analysis of a flexible conduit is a study of the ring compression strength and ring bending strength. The ring compression theory illustrates the interaction of the conduit and the surrounding soil which together support the applied loads.

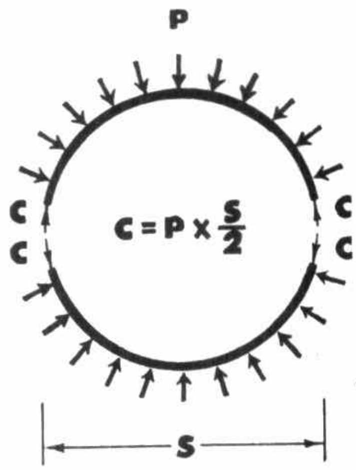
Soil analysis should include three design checks.

1. classification and evaluation of soil type on site or to be used as backfill material.
2. determination of fill soil properties including the modulus of deformation or the compaction characteristics of the soil and the coefficient of the soil reaction.
3. determination of subgrade soil properties including consolidation tests.

Field measurements known as Farina tests have shown that a flexible conduit bears about 60% of the load of the soil prism above the pipe while the soil withstands the remaining 40%.

I Ring Compression

Figure 3 is a simple body diagram showing that the value of conduit wall compression thrust (lbs/ft of culvert) is equal to the sum of the dead loads and live loads (in psf) on the top of the pipe multiplied by one-half of the maximum horizontal span (ft.)



Formula for ring compression design.

Fig. 3

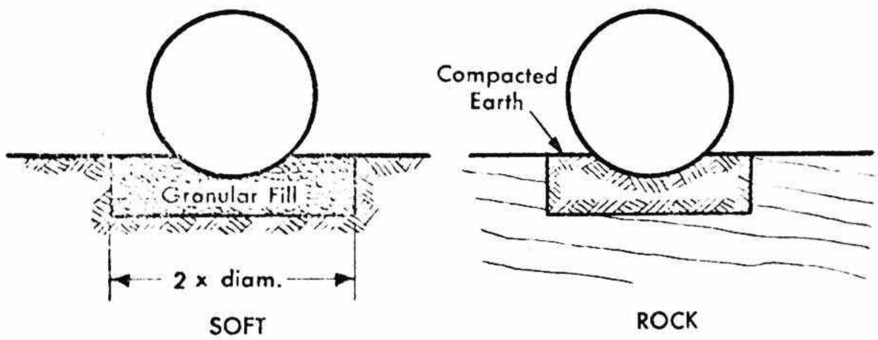
1. Dead Loads

To determine the value of the dead load acting on the top of the conduit, the designer should assume that this load is equal to the height of fill times its density. In actual cases, however, such as where soft foundations are encountered, the dead load on the conduit is reduced. Conversely, when the foundation beneath the conduit is more firm relative to the area on each side of the pipe, the load is increased. This is analogous to the review of the Marston formula for loads on rigid pipe where friction planes of the side earth prisms relieve or increase the load on the conduit.

Figure 4 illustrates the correct procedure to be followed when installing flexible pipe in soft or hard foundation materials. By adopting this method a uniform foundation is provided under the

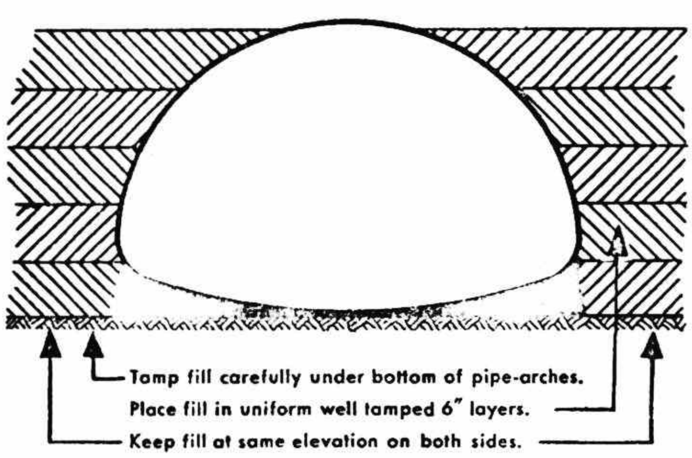
structure as well as at each side.

Figure 5 shows that when some degree of compaction is provided at the sides of the structure the deflection will be a minimum and the load on the structure will be equal to that calculated from the height of cover under higher fills.



Left: Unstable foundation material should be removed below flow grade line and replaced with sand, gravel, crushed stone or other suitable material. *Right:* When rock is encountered in the foundation, it should be excavated to at least 8 in. below the bottom of the structure.

Fig. 4



Compaction under the corners of pipe-arches is important.

Fig. 5

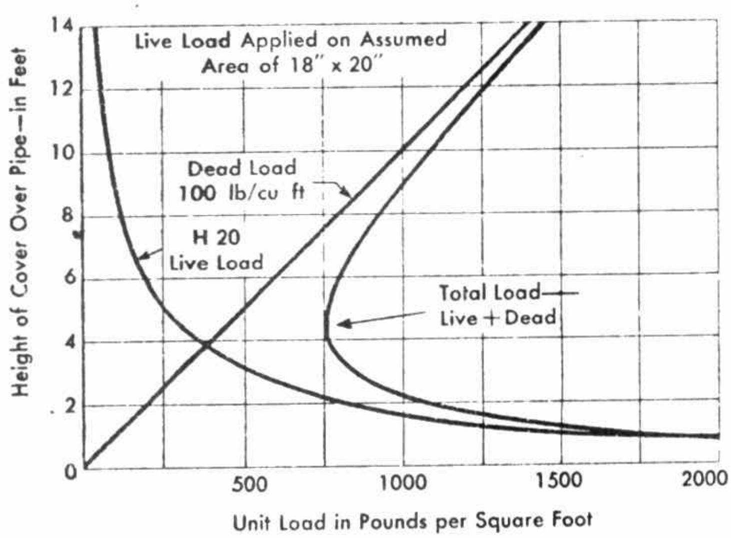
2. Live Loads

The magnitude of the load live acting on a buried conduit varies inversely as the depth of fill. There are various theories available to the designer, the Boussinesq solution being the most widely used.

Figures 6 and 7 show the graphical relationship of the distribution of live load to depth of cover for a H-20 highway loading and for a Cooper E-72 railroad live load respectively.

The H-20 loading represents a 20 ton vehicle, having a rear axle load of 80% of the weight of the vehicle and the load is applied on a pressure area of 18" x 20" (area of contact between the tires and the road surface).

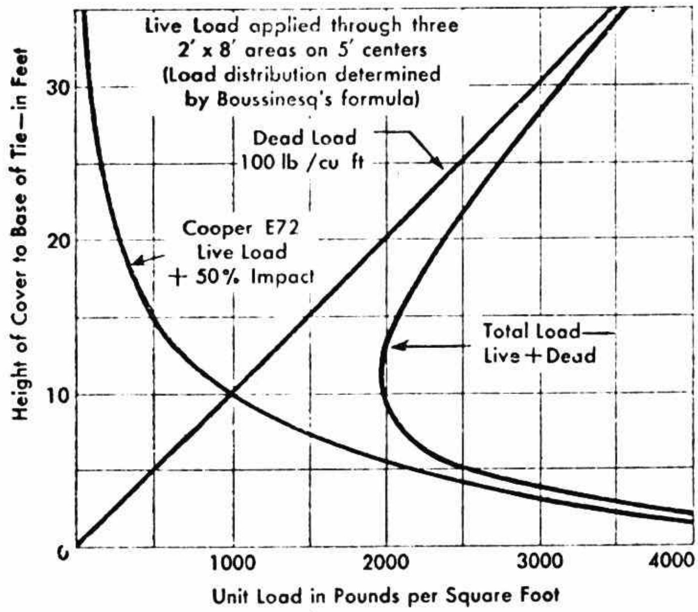
The Cooper E-72 loading is the live load transmitted to a pipe due to the weight of the locomotive driver axles plus the weight of the track structure, including ballast. This load is assumed to be uniformly distributed over an area equal to the length of the ties multiplied by the length occupied by the driver axles.



Combined H 20 highway live load and dead load is a minimum at about 4 ft of cover, applied through a pavement 1 ft thick.

Fig. 6

TABLE OF CONTENTS



Railroad live load, Cooper E 72, combined with dead load, is a minimum at about 11 ft. Load is applied through three 2 ft by 8 ft areas on 5-ft centers.

Fig. 7

3. Impact Factors

(a) - Highway loading - The surface live load should be increased by an impact factor, which varies between 1.0 and 1.3 for depths of cover ranging from 3' to 0' respectively.

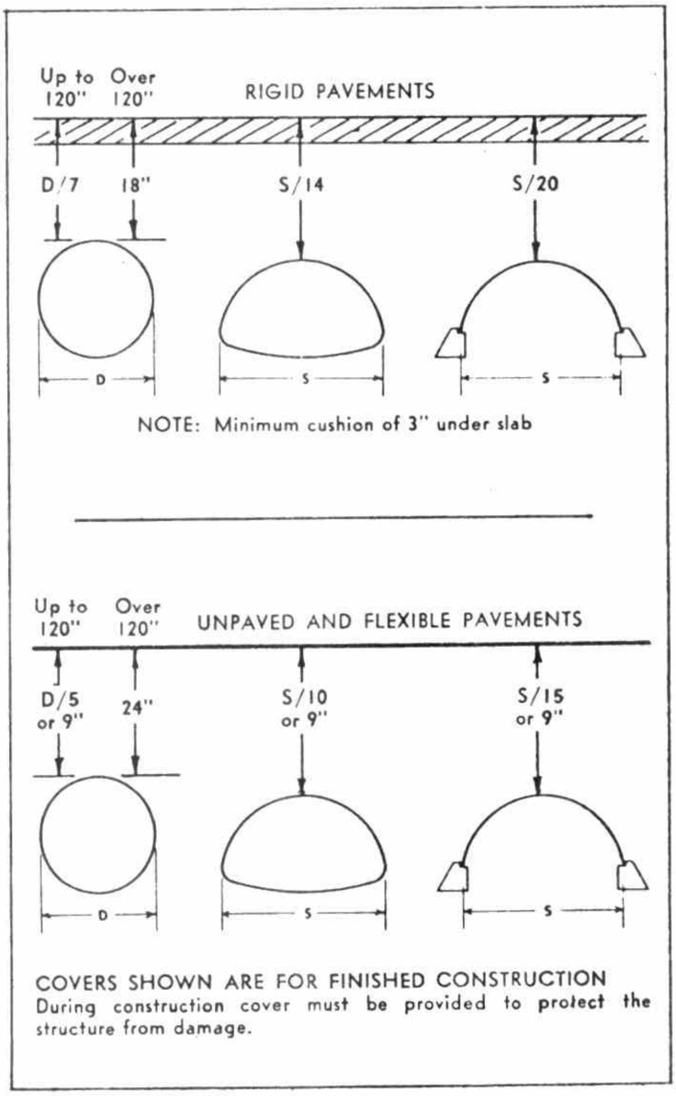
(b) - Railroad loading - The American Railway Engineering Association recommends that an impact factor of 1.0 to 1.5 be used for depths of cover between 10' and 0' respectively.

4. Application of Live Load

The designer should be aware that in certain cases it may be possible that more than one wheel load is transmitting load to the conduit. This would double the live load on the pipe as determined from figure 6.

5. Minimum Height of Cover

The minimum recommended allowable height of cover is given in figure 8 below.



Minimum height of cover for corrugated metal structures. Final or finished grade.

Fig. 8

At the point of minimum cover, the normal components of the active pressures on a structure using the active component of the vertical weight as one-third, (Rankine's ratio) shows this

point to be approximately one-quarter of the diameter of a round pipe as shown in figure 8. . In practical applications for highway loading this can be reduced to at least one-eighth the diameter because the structure does contain moment strength. Railroad loading should be designed for a minimum cover equal to one-quarter of the diameter of the conduit. Experiments are still being carried out in this field to refine this information.

II Bending Strength

The second aspect which the designer must consider is the ring bending strength. This can be broken down into three sections: seam strength, buckling strength, and installation rigidity.

1. Seam Strength

For the design of seam strength, the conduit wall compression is limited to the ultimate strength of the seam divided by a factor of safety.

Table 11 gives the values of the ultimate seam strength for various sizes of corrugated metal pipe.

Insufficient research exists in the determination of the yield point causing a failure by compressive buckling. However, based on vast experience, it has been found that a safety factor of 2 can be used for carefully controlled and well engineered installations. However, it is recommended that a factor of safety of 4 be used in determining seam strength for average handbook-type installation.

Ultimate Longitudinal Seam Strength of Corrugated Steel Pipe
In Pounds Per Foot of Seam

Gage of Metal	5/16" Rivets		3/8" Rivets			7/16" Rivets
	2⅔" x ½"		2⅔" x ½"		3" x 1"	3" x 1"
	Single	Double	Single	Double	Double	Double
16	16,750	21,500			28,700	
14	18,200	29,800			35,700	
12			23,400	46,800		53,000
10			24,500	49,000		63,700
8			25,600	51,300		70,700

The above values are based on riveted construction; they also apply to spotwelded and helical lock seam fabrication.
Values in this table are based on tests conducted by Utah State Dept. of Highways, 1964, and by Pittsburgh Laboratories, 1966.

Ultimate Strength of Bolted Structural Plate Longitudinal Seams
In Pounds Per Foot of Seam

Gage	4 Bolts Per Foot	6 Bolts Per Foot	8 Bolts Per Foot
12	42,000		
10	62,000		
8	81,000		
7	93,000		
5	112,000		
3	132,000		
1	144,000	184,000	220,000

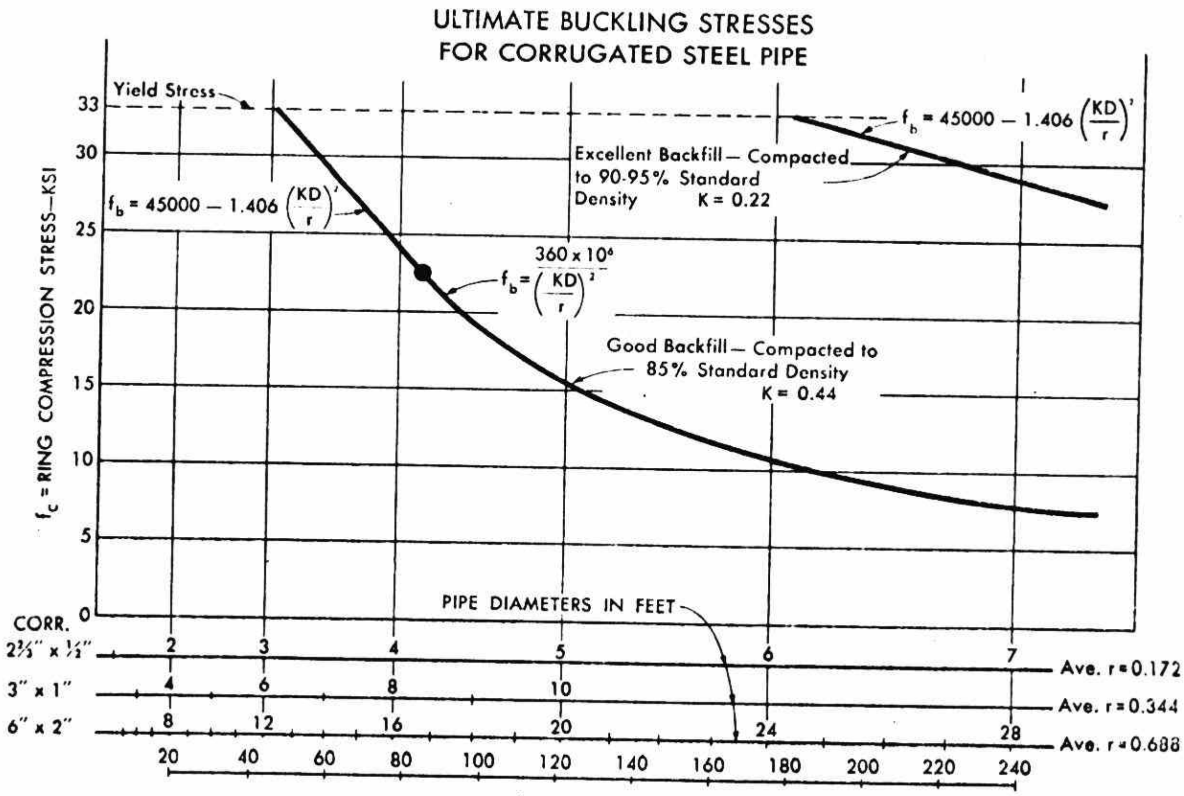
Bolts used in tests were ¾-in. high strength bolts, meeting ASTM A 325.

TABLE II

2. Buckling Strength

The structure is designed for buckling by limiting the ring compression to the stress at buckling multiplied by the cross-sectional area of the conduit wall divided by the factor of safety.

Figure 9 illustrates the relationship of ring compression to ultimate buckling stresses for good and excellent backfill conditions.



Ultimate buckling stresses for two types of backfill ("good" and "excellent").

Fig. 9

The cross sectional area of corrugated metal pipe walls is tabulated below.

Moment of Inertia and Cross Sectional Wall Area of Corrugated Steel Sheets and Plates for Drainage Conduits

Corrugation Dimensions	Moment of Inertia, I in Inches^4 per Inch of Width								
Pitch x Depth	16 Ga	14 Ga	12 Ga	10 Ga	8 Ga	7 Ga	5 Ga	3 Ga	1 Ga
1½ x ¼ in.	.00044	.00057	.00086	.00121	.00163				
2 x ½ in.	.00194	.00246	.00354	.00472	.00599				
2⅔ x ½ in.	.00189	.00239	.00343	.00453	.00572				
3 x 1 in.	.00866	.01088	.01546	.02018	.0251				
6 x 2 in.			.0604	.0782	.0961	.1080	.1269	.1462	.1659

Cross Sectional Wall Area, in Inches^2 per Inch of Width

1½ x ¼ in.	.0634	.0792	.1109	.1427	.1744				
2 x ½ in.	.0679	.0849	.1190	.1532	.1874				
2⅔ x ½ in.	.0646	.0807	.1130	.1453	.1777				
3 x 1 in.	.0742	.0927	.1300	.1673	.2048				
6 x 2 in.			.130	.167	.204	.228	.267	.305	.343

Corrugation dimensions are nominal; subject to manufacturing tolerances.

TABLE III

The recommended factor of safety for design by buckling is 2 based on numerous tests and experience.

3. Handling and Installation

Flexible conduits must have adequate rigidity to maintain their shape for handling, shipping and installation.

This flexibility factor or relative elastic deflection factor can be expressed as F.F. = D^2/EI

where D = dia. or max. span of conduit (in.)

E = modulus of elasticity of pipe material (psi)

I = moment of inertia per unit length of cross section of the pipe wall (in^4)

No theoretical value of the flexibility factor is available; however, based on experience empirical maximum tolerable values for steel conduits are as follows:

for 2-2/3"	x 1/2"	corrugations	F.F.	0.0433
for 3"	x 1"	corrugations	F.F.	0.0433
for 6"	x 2"	corrugations	F.F.	0.0200

4. Deflection

Flexible pipe fail by excessive deflection and collapse rather than rupture of the pipe walls as in the case of rigid conduits. Ring deflection then is the change in diameter (horizontal as well as vertical) due to flattening under an applied load. If the deflection exceeds 5% of the nominal diameter of the conduit, collapse will ensue. Since deflection indicates inadequate compaction, an A.A.S.H.O. density of the backfill of 90% to 95% is recommended.

Conduit deflection can be determined by the Iowa Deflection Formula,

$$\Delta_x = D_1 \frac{KWcR^3}{EI + 0.061E^1R^3}$$

where Δ_x = horizontal and vertical deflection of the pipe (in.)

D_1 = deflection lag factor

K = a bedding constant dependent upon the angle subtended by the pipe bedding.

Wc = vertical load on the pipe (lb. per lin. in.)

R = mean radius of the pipe (in.)

E = modulus of elasticity of the pipe material (p.s.i.)

I = moment of inertia per unit length of cross section of pipe wall (in^4 per in.)

E^1 = eR = modulus of soil reaction (psi)

e = modulus of passive resistance of the enveloping soil (psi)

The above formula relates pipe deflection to passive side pressure resisting horizontal movement of the pipe and to the inherent strength of the pipe and has been derived from analysis and study of existing structures. The designer can also be confident of its reliability in using this formula for the design of new structures.

It should be noted that there is little information available on the value of the soil modulus, E'. This factor has not been accurately correlated with the backfill type and degree of compaction.

Recommended values for the various factors used in the computation of the deflection of a given culvert are as follows.

A. Good Backfill Material - 85% Proctor Density.

D_1	K	E^1	Remarks
1.5	0.44	700 psi	is based on an average of actual installations

B. Excellent Backfill Material - 95% Proctor Density.

D_1	K	E^1	Remarks
1.25	0.22	1400 psi	is based on an average of actual installations

Not all the deflection occurs instantaneously upon the load being applied! Therefore, the lag factor D_1 considers the time delay.

It should be noted that some texts consider the factor, K to vary between 0.083 and 0.110 for a bedding angle of 180^o and 0^o respectively. In the tables above the most recent published values for this factor have been quoted.

The influence of the inherent strength of the pipe on deflection, expressed by the first term of the denominator, should not be less than 10-15% than the value of the second term of the denominator, $0.061\ E'R^3$. This is particularly true when analyzing large structures where the passive earth pressure is the governing portion in resisting deflection. The designer must keep in mind, however, the fact that the pipe wall must have sufficient local

strength in bending and thrust to develop and utilize the passive resistance pressure on the sides of the pipe.

The following problem illustrates how the above design factors determine the selection of a flexible conduit.

Figure 10 shows a summary of the types of failure which are possible in flexible conduit installations.

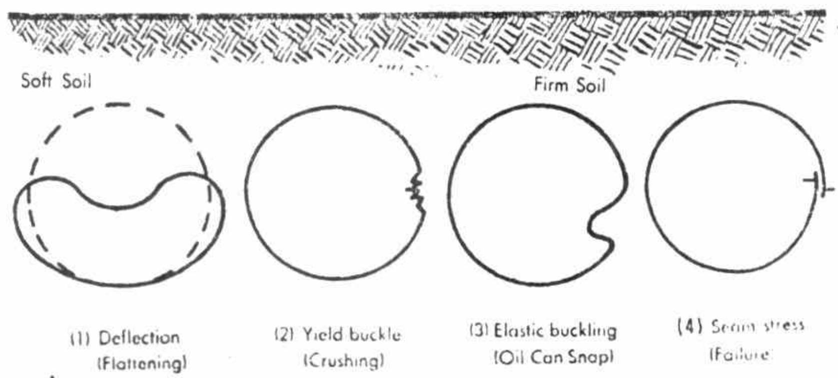


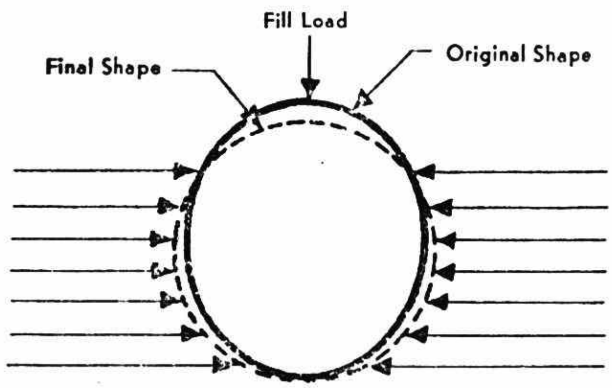
Fig. 10

III

1. <u>Vertical Elongation</u>

Elongating the vertical diameter of the pipe <u>before backfilling</u> allows additional side support to be built up as the pipe resumes a full-round shape under a heavy fill load. In this manner the load carrying capacity is increased over that for a pipe installed with a round shape. The amount of elongation is usually equal to 5% of the nominal diameter of the pipe. It should be noted that this procedure is required only on large conduits under high fills. Elongation can be achieved in the manufacture of the plate itself or by wire strutting, applying a rod and turnbuckle or by strutting.

Figure 11 illustrates that as the load is applied and deflection occurs, the backfill on the sides of the pipe is compressed and therefore the side support for the structure is increased. Small deflections indicate excellent backfilling.



Proper side support enables a flexible steel structure to resist tremendous fill loads.

Fig. 11

2. Backfilling

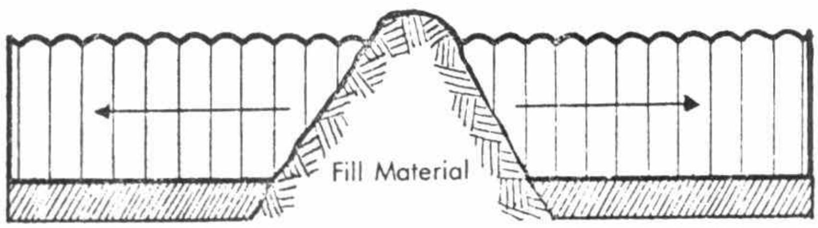
Figure 12 shows the recommended and poor backfilling practice for corrugated metal pipe arches. This procedure can also be applied to circular pipe installations.

Filling on only one side causes arch to shift. If fill is not placed on top as backfilling proceeds, arch may raise, thereby flattening side radius.

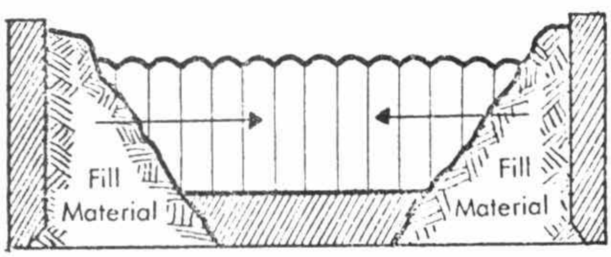
COMMON MISTAKES IN BACKFILLING ARCHES (above)

Place fill on arch by distributing material around and over the structure in uniform layers, tamping thoroughly. Place material from top of arch.

RECOMMENDED BACKFILLING PRACTICE



SIDE VIEWS—Without and With Headwalls



Recommended and poor backfilling practice for corrugated steel arches. *

Fig. 12

Corrugated metal pipes should not be placed on soft or hard foundations or on timber or concrete cradles or piles and should not be encased in concrete.

The designer should be completely familiar with the texts available on the subject of installation of flexible conduits since it is only through achievement of adequate field control that the proper specifications are fulfilled and the design of the structure is realized.

3. Height of Cover Tables

Tables IV and V tabulate the limitations in height of cover for various pipe sizes and gauge.

Note that a 48" diameter pipe of 16 gauge can be installed at a depth of 22 feet while if the same conduit was made with 8 gauge walls then it could withstand a depth of 26 feet. Further this same pipe is adequate for depths of 27 feet and 52 feet for 16 and 8 gauge respectively if 5% vertical elongation is permitted. Note the vast increase of this latter allowable limit over the value of the round pipe!

TABLE IV & V

Height-of-Cover Limits for Corrugated Steel Pipe in Feet, for H20 Loading

2⅔ in. x ½ in. Corrugations

Riveted, Spot Welded or Helical Lock Seam Fabrication

K = 0.44 (Good Backfill)

Diameter or Span—Inches	16 Gage		14 Gage		12 Gage		10 Gage		8 Gage	
	Round	Elong*	Round	Elong*	Round	Elong*	Round	Elong*	Round	Elong*
12	83		91							
15	67		72							
18	55		60							
21	44	—	50	—	63	—				
24	36	—	40	—	48	—				
30	28	—	30	—	35	—				
36	25	28	26	30	28	39	31	40	—	—
42	23	31	24	42	25	50	27	54	29	58
48	22	27	22	37	24	48	25	50	26	52
54	—	—	22	33	22	44	23	46	24	48
60			—	—	22	42	22	44	23	46
66			—	—	21	31	22	42	22	44
72					—	—	21	32	22	41
78					—	—	—	—	21	32
84					—	—	—	—	21	25

Minimum cover top of pipe to top of subgrade = 1 foot

*Vertical elongation = 5% of nominal diameter

DESIGN CRITERIA

Dead Load Pressure—100 psf per foot of height of cover.

Seam strength based on Table 2-4.

Rivet pattern:

12 thru 36 in. diameter—4½ rivets per foot

42 thru 84 in. diameter—9 rivets per foot

16 and 14 gage—5/16 in. diameter rivets

12, 10 and 8 gage—⅜ in. diameter rivets

Safety factors based on:

FS = 4 for longitudinal seams

FS = 2 for pipe wall buckling

FF = .0433; maximum span limited by flexibility factor

Good backfill material compacted to 85% Proctor density.

K = 0.44; soil coefficient

E' = 700 psi; modulus of soil reaction

Δ_x = 5% below circular shape

Refer to text for design procedure. Fill heights exceeding 100 ft shall be considered a special design.

Height-of-Cover Limits for Corrugated Steel Pipe in Feet, for H20 Loading

2⅔ in. x ½ in. Corrugations

Riveted, Spot Welded and Helical Lock Seam Fabrication

K = 0.22 (Excellent Backfill)

Diameter or Span Inches	16 Gage		14 Gage		12 Gage		10 Gage		8 Gage	
	Round	Elong*	Round	Elong*	Round	Elong*	Round	Elong*	Round	Elong*
12	83	—	91	—	—					
15	67	—	72	—	—					
18	55	—	60	—	—					
21	47	—	52	—	66					
24	41	—	45	—	58					
30	33	—	36	—	46					
36	27	28	30	30	39	39	40	40	—	—
42	30	31	42	42	46	66	48	69	49	73
48	26	27	37	37	44	58	45	61	46	64
54	—	—	33	33	43	52	44	54	45	57
60					42	46	43	48	44	51
66					42	42	42	44	43	46
72					—	—	40	40	42	42
78					—	—	—	—	39	39
84					—	—	—	—	36	36

Minimum cover top of pipe to top of subgrade = 1 foot

*Vertical elongation = 5% of nominal diameter

DESIGN CRITERIA

Dead Load Pressure—100 psf per foot of height of cover.

Seam strength based on Table 2-4.

Rivet pattern:

12 thru 36 in. diameter—4½ rivets per foot

42 thru 84 in. diameter—9 rivets per foot

16 and 14 gage—5/16 in. diameter rivets

12, 10 and 8 gage—⅜ in. diameter rivets

Safety factors based on:

FS = 4 longitudinal seams

FS = 2 for pipe wall buckling

FF = .0433; maximum span limited by flexibility factor

Excellent backfill material compacted to 95% Proctor density.

K = 0.22, soil coefficient

E' = 1400 psi; modulus of soil reaction

Δ_x = 5% below circular shape

Refer to text for design procedure. Fill heights exceeding 100 ft shall be considered a special design.

PROBLEM - Flexible Conduit

Given: 36" dia. CMP 2-2/3" x 1/2" corr. riveted (5/16") single

H= 3' (above crown)

Find: Required Gauge.

Solution:

Ring Compression: $C = \frac{PS}{2} = (D.L. + L.L.)\frac{Span}{2}$

D.L. = dead load of soil prism above conduit
= WH = 100 x 3 = 300 psf

L.L. = 600 psf from Fig. 6

$C = (300 + 600)\frac{3}{2} = 1350$ p.l.f.

Seam Strength Required = F.S. x C

use F.S. = 4

$\therefore$ Seam strength required = 4 x 1350 = 5400 p.l.f.
16 Ga. has seam str = 16750 plf.
Table II

Buckling of Wall

assume good backfill

from figure 9 ultimate buckling strength f_b = 33000 psi

f_b allowable = $\frac{f_{b_{ult}}}{F.S.}$ F.S. in buckling = 2

f_b = working buckling stress

f_c = compressive stress

A = area of pipe wall in^2/in - for 16 guage
= 0.0646 in^2/in TABLE III

C = 1350 p.l.f.

$f_c = \frac{C}{12A} = \frac{1350}{12 \times 0.0646}$ = 1741 p.s.i. $\frac{f_{b_{ult}}}{F.S.} = \frac{33000}{2}$ = 16500 p.s.i.

$\therefore$ OK for buckling

Handling & Installation

$$FF = \frac{D^2}{EI} = \frac{36^2}{(30 \times 10^6)\ (0.00189)} = 0.0216$$

which is less than allowable FF = 0.0433

N.B. I read from table III or $(I = \frac{bd^3}{12})$

Deflection

$$\Delta x = \frac{D.\ K\ Wc\ R^3}{EI + 0.061\ E'R^3}$$

$$Wc = \frac{(DL + L.L.)}{12}\ SPAN = \frac{(300 + 600)}{12}\ 3 = 225\ \text{lb/in}$$

I (from table III) = 0.00189 in^4 (mom. of inertia)

$E = 30 \times 10^6$ psi. (mod. of elast.)

$R = 18''$ (radius of pipe)

$K = 0.44$ (bedding factor))
)
$E' = 700$ psi. horiz.soil mod.)

Assuming only
Good Compaction

For good compaction:

$$\Delta x = \frac{1.5 \times .44 \times 225 \times 18^3}{(30 \times 10^6) \times .00189 + (.061 \times 700 \times 5820)}$$

$$= \frac{.865}{.306} = 2.93''$$

Allow deflection = 5% of vert. dia.

= 0.05 x 36 = 1.80 in.

∴ N.G.

Try excellent compaction

$$\Delta x = \frac{1.25 \times 0.22 \times 225 \times 18^3}{(30 \times 10^6 \times .00189) + (.061 \times 1400 \times 18^3)}$$

$$= \frac{.36}{.555} = .65\ \text{in.}$$

$< \Delta x$ allow

∴ OK for excellent backfill

Check ratio of inherent strength to soil resistance

$$\frac{EI}{.061\ E'R^3} = \frac{30 \times 10^6 \times .00189}{.061 \times 1400 \times 18^3} = 11.4\%$$

which is in the recommended range.

Summary - A 36" dia. CMP @ 3'-0 of cover requires a 16 gauge material and excellent backfill (95% Std. Proctor Density).

Earth Loads on Trench Conduits

Trench conduits are installed in relatively narrow trenches excavated in undisturbed soil and then covered with earth backfill which extends to the original ground surface.

1. Marston Formula - Theory

Earth loads on buried pipes can be determined by the Marston equation, $Wd = Cd \; w \; Bd^2$

where Wd = vertical earth load acting on the pipe (plf).

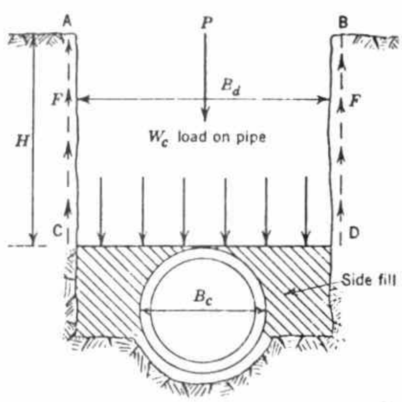
Cd = load coefficient for trench condition

w = unit weight of backfill material

Bd = width of trench at top of conduit (ft.)

The theory, based on experiments, states that the load on a buried conduit is equal to the actual weight of the prism of earth directly over the pipe plus or minus the frictional shearing forces on this prism created by the relative movement of the prisms of earth on each side of the conduit.

Figure 13 illustrates this relationship.



—LOAD-PRODUCING FORCES: P = WEIGHT OF BACKFILL ABCD; F = UPWARD SHEARING FORCES ON AC AND BD, AND $W_c = P - 2F$

Fig. 13

The assumptions on which this theory is based are:

1. The load on the conduit develops as the backfill settles.
2. The resulting load on the conduit is equal to the weight of the material above the top of the conduit minus the shearing or frictional forces on the side of the trench.
3. The magnitude of the lateral earth pressures which induce the shearing forces between the interior and exterior earth prisms is computed in accordance with Rankines Theory.
4. Cohesion is assumed to be negligible except for tunnels. There is a time lag factor here; a more conservative design will result by ignoring it.

2. Coefficient, Cd

Figure 14 is a graphical analysis of the relationship of various soils types, heights of cover and widths of trench.

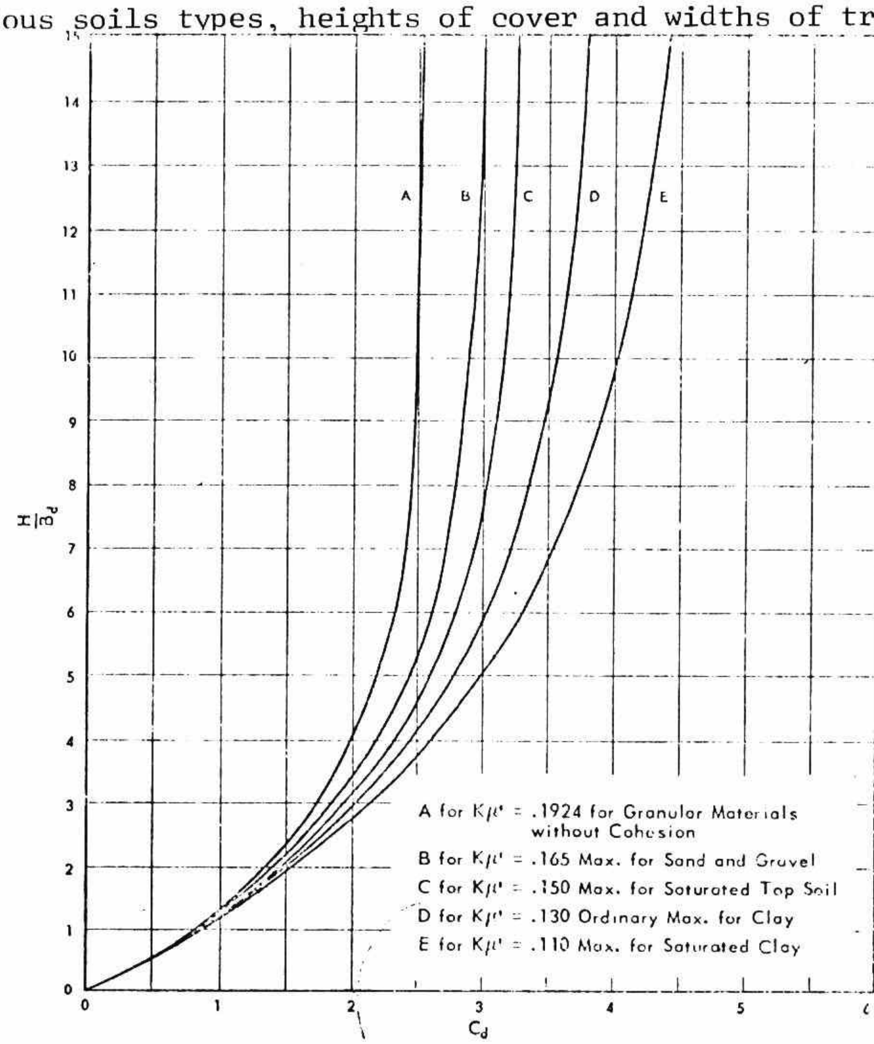


Fig. 14

The designer should use curve D if no other information is available.

If sheeting is left in place the designer should be aware that the coefficient of friction, μ between the timber and the adjacent ground is less than the value for the case where the sheeting is removed. Therefore the coefficient Cd is increased which also increases the load on the pipe.

3. Unit Weight, w

If soil tests have not been taken a conservative assumption for design purposes for the value of the unit weight of backfill material is 120 p.c.f.

4. Width of Trench, Bd

The width of trench, as seen above, is taken at the top of the conduit. This factor, more than any other in the Marston equation, influences the magnitude of the calculated earth load on the conduit.

The designer's attention is drawn to Figure 15 which shows this increase in load with all other factors remaining the same.

Therefore, the most economical design is to restrict the width of trench to the minimum allowable under practical construction procedures. Note that sloping or widening the trench walls above the top of the conduit has no effect on the magnitude of the earth load on the pipe.

If sheeting is used the width of trench shall be taken as the overall excavated width, including the sheeting unless the sheeting is left in place.

By definition, the Marston Theory states that the trench conduit is installed in narrow trenches. In practice, this has been found to be up to approximately 2.5 times the diameter of the pipe. When a pipe is installed in a trench having a width exceeding this measurement

the load on the conduit must be calculated by the Embankment Conduit method. Figure15 plots the relationship of the ratios H/Bc and Bd/Bc for various values of rsdp. The term rsdp is assumed to have a value of 0.5 x 0.7 = 0.35 if no other data is available. This term will be discussed later in this report.

APPROXIMATE TRANSITION RATIO CURVES FOR PIPE CONDUITS IN WIDE TRENCHES BACKFILLED WITH HOMOGENEOUS, ELASTIC, GRANULAR, SLIGHTLY COHESIVE, SOIL MEDIUMS, COMPACTED TO THEIR MAXIMUM DENSITY.

(For various numerical values of $r_{sd}p$)

very nearly linear extrapolation of curve.

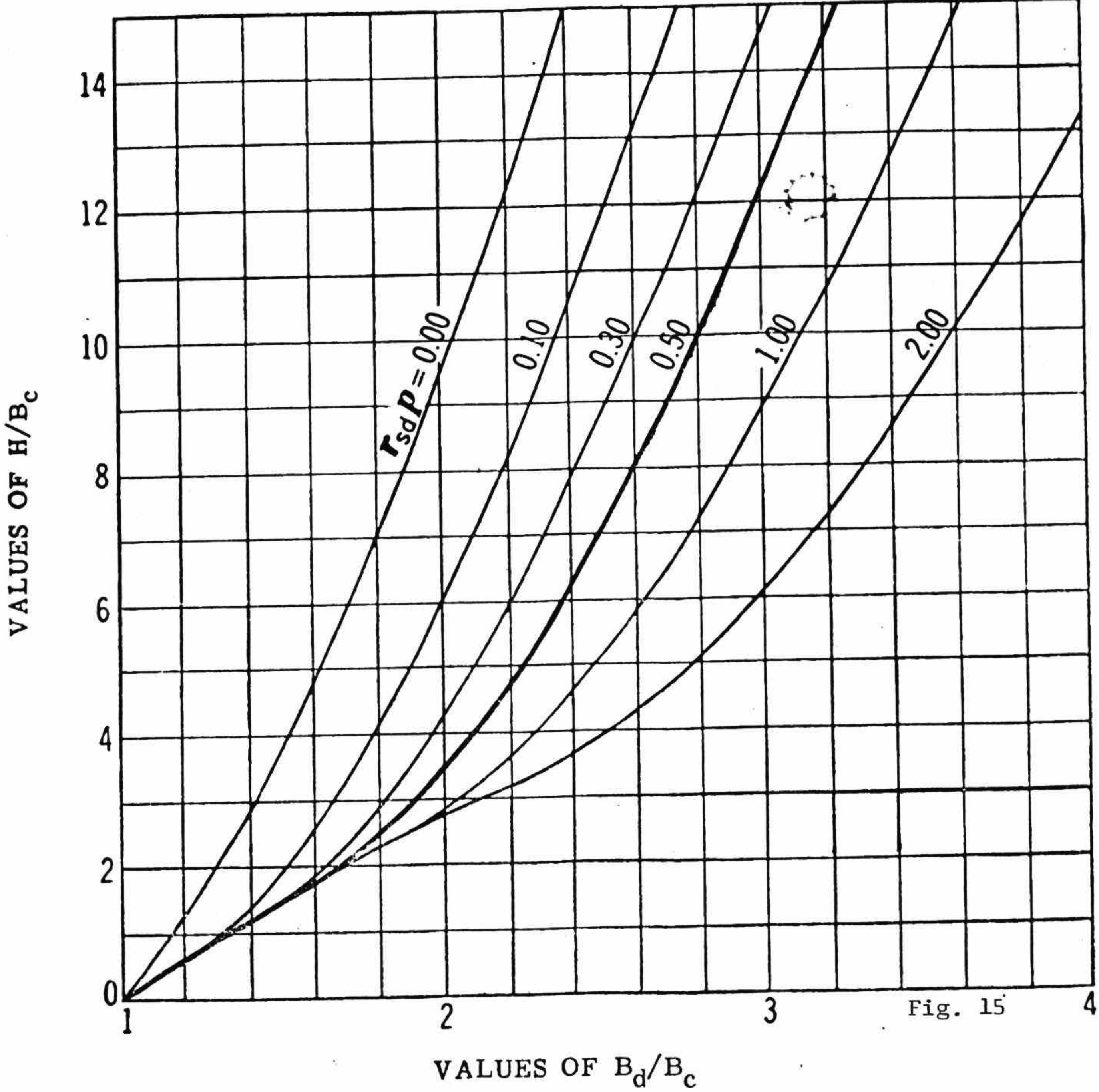
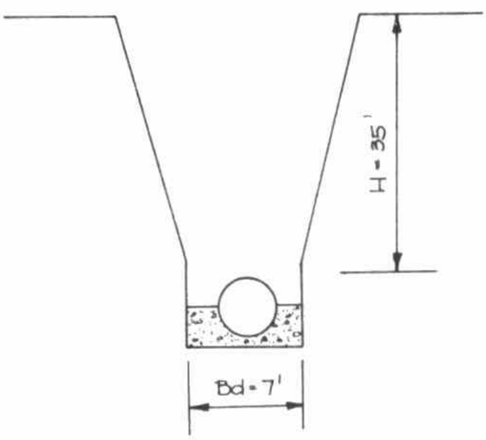


Fig. 15

PROBLEM #1 - Trench Conduit

Given : A 48" dia. circular pipe is to be installed in a 7-foot wide trench with 35 feet of cover over the top of the pipe. The pipe will be backfilled with sand and gravel weighing 110 lbs. per cubic foot.



Required: Class of Pipe required.

Solution: Assume Class B bedding - stone etc.

$Bc = 4 + 2\ (.42) = 4.84'$

$H/Bc = 35/4.84 = 7.23$

from chart 15 & for $r_{sd}\,p = +\ 0.35$

where $r_{sd} = +\ 0.5$ to $+\ 0.8$ for ordinary foundations

& $p = 0.7$ assumed.

Transition ratio $Bd/Bc = 2.42$

$\therefore$ Transition width $= 2.42 \times 4.84 = 11.5'$

Since the above pipe is being installed in a trench having a width of 7' $\therefore$ trench conditions prevail.

for Class B bedding $Lf = 1.9$ (from chart 29)

$Wc = Cd\ w\ Bd^2$ where Wc = load on pipe (plf).

Cd = load calculation coefficient

w = unit wt. of backfill soil

Bd - width of trench

determine Cd from graph # 14 for sand and gravel backfill (curve B) & $H/Bd = 35/7 = 5$

$Cd = 2.4$ (From Page 29)

$Wc = 2.4 \times 110 \times 7^2 = 12,930$ plf.

For reinf. concrete pipe S.F. = 1.0

$\therefore$ Safe supporting strength = $\dfrac{\text{field supporting strength}}{\text{factor of safety}}$

$= \dfrac{12930}{1.0} = 12,930$ plf.

Since field supporting strength = 3 edge brg.str. x load factor

$\therefore$ Req'd 3 edge brg.strength = $\dfrac{\text{max.field load x factor of safety}}{\text{load factor}}$

$= \dfrac{12930 \times 1.0}{1.9} = 6810$ plf.

req'd D load to produce a 0.01" crack

= Req'd 3 edge brg.strength/int.dia.of pipe

= 6810/4 = 1702 plf/foot of dia.

from pipe spec's. ASTM C-76-68 (Page 5)

need Class IV which has a D load of

2000 plf/ft. of dia.

Earth Loads on Embankment Conduits

Embankment conduits, as the name infers, are those placed under fills or embankments. Trench conduits having extremely wide trenches are also to be included in this category.

Figure 1 of the introduction illustrates the three types of embankment conduits, the positive projecting, negative projecting and induced trench. Each of these classifications will be very briefly explained as follows.

Positive Projecting Conduit

This condition, as illustrated in figures 16 & 17 occurs when a pipe is installed in shallow bedding with the top of the pipe projecting above the surface of the original ground or compacted fill at the time of construction, and then covered with earth fill.

Settlements Which Influence Loads on Embankment Conduits — Incomplete Projecting Condition

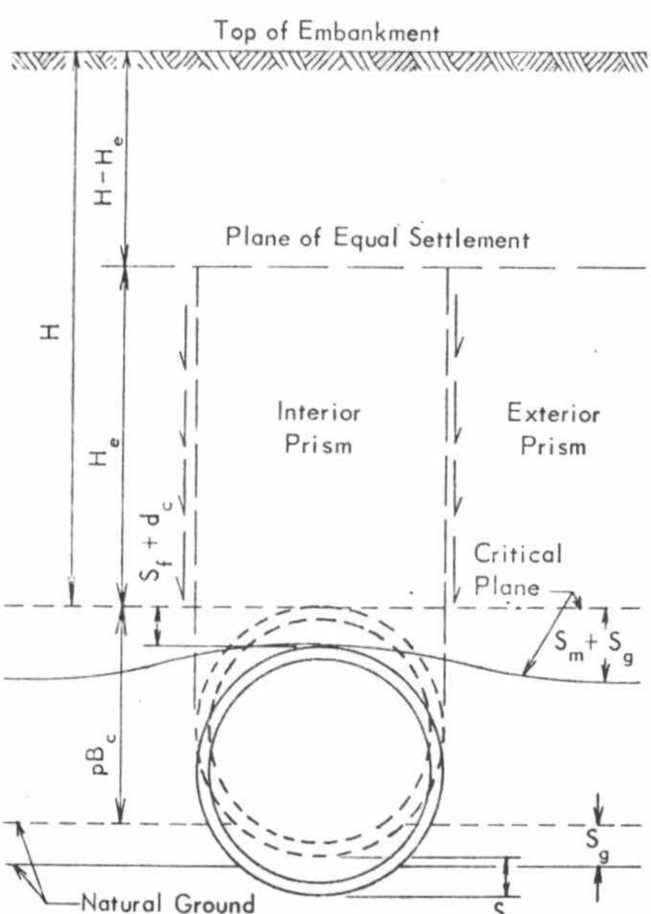


Fig. 16

Settlements Which Influence Loads on Embankment Conduits — Incomplete Trench Condition

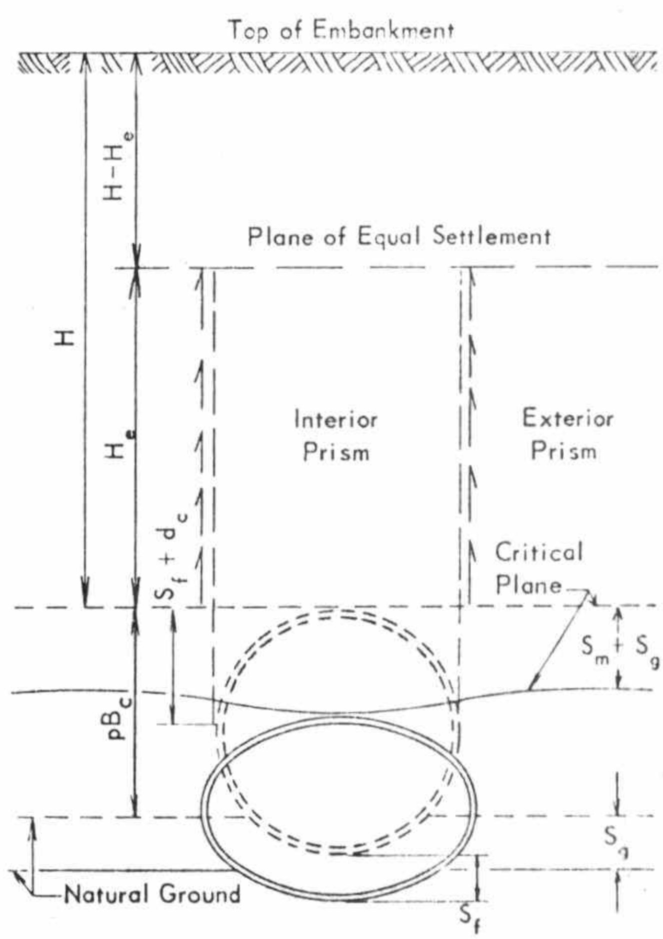


Fig. 17

The load on the conduit can be calculated by the formula -

$$Wc = Cc\ W\ B_c^2$$

where Wc = earth load on pipe p.l.f.

Cc = load coefficient for positive projecting embankment conduit.

W = unit weight of backfill p.c.f.

Bc = outside horizontal diameter of conduit (ft).

There are two ways in which the load can act on the pipe dependent upon the relative settlement of the pipe and the adjacent earth prisms. Figure 16 illustrates the case where the load on the pipe is greater than the dead load of earth above the top of the conduit.

In Figure 17 the frictional forces relieve some of the total dead load acting on the top of the pipe.

Load Coefficient, Cc

This coefficient is dependent upon the ratio of height of fill to horizontal width of conduit, H/Bc, the coefficient of internal friction of the soil, μ, the projection ratio, p, and the settlement ratio, r_{sd}.

For design purposes a value of 0.6 is recommended for the coefficient of internal friction of the soil, μ .

The projection ratio, p , can be defined as the vertical distance between the top of the pipe and the original ground divided by the outside horizontal diameter of the pipe.

Unless detailed information is available, it is safe to assume a value of 0.7 for the projection ratio, p .

The settlement ratio, rsd , is the ratio of the settlement of the earth prisms adjacent to the conduit, relative to the settlement and deflection of the conduit divided by the compression of the exterior prism. This is more clearly explained by the formula

$$r_{sd} = \frac{(S_m + S_g) - (S_f + d_c)}{S_m}$$

where rsd = settlement ratio

Sm = compression of exterior prisms of soil of height pBc

Sg = settlement of original ground or compacted fill adjacent to the conduit.

Sf = settlement of conduit into its bedding foundation.

dc = deflection of the vertical height of the conduit.

Figure 18 further serves to show this relationship.

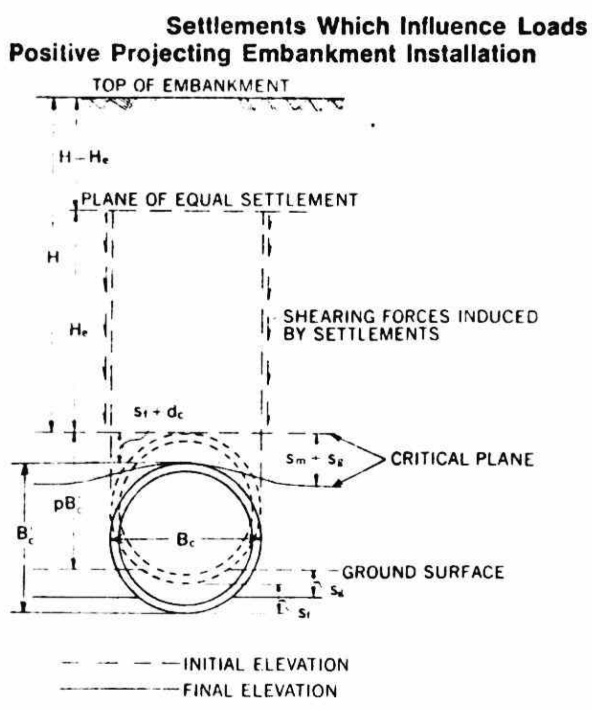


Fig. 18

The range of the settlement ratio is given in table VI together with the recommended design values for varying soil types.

DESIGN VALUES OF SETTLEMENT RATIO

Installation and Foundation Condition	Settlement Ratio r_{sd} Usual Range	Settlement Ratio r_{sd} Design Value
Positive Projecting	0.0 to +1.0	
Rock or Unyielding Soil	+1.0	+1.0
Ordinary Soil	+0.5 to +0.8	+0.7*
Yielding Soil	0.0 to +0.5	+0.3
Zero Projecting		0.0
Negative Projecting	-1.0 to 0.0	
p' = 0.5		-0.1
p' = 1.0		-0.3
p' = 1.5		-0.5
p' = 2.0		-1.0
Induced Trench	-2.0 to 0.0	
p' = 0.5		-0.5
p' = 1.0		-0.7
p' = 1.5		-1.0
p' = 2.0		-2.0

*N.B.

TABLE VI

* N.B. - other sources more recent recommend + 0.5 and this is what is used in this article!

The load coefficient can be determined from the graphical relationship shown in fig. 19 below.

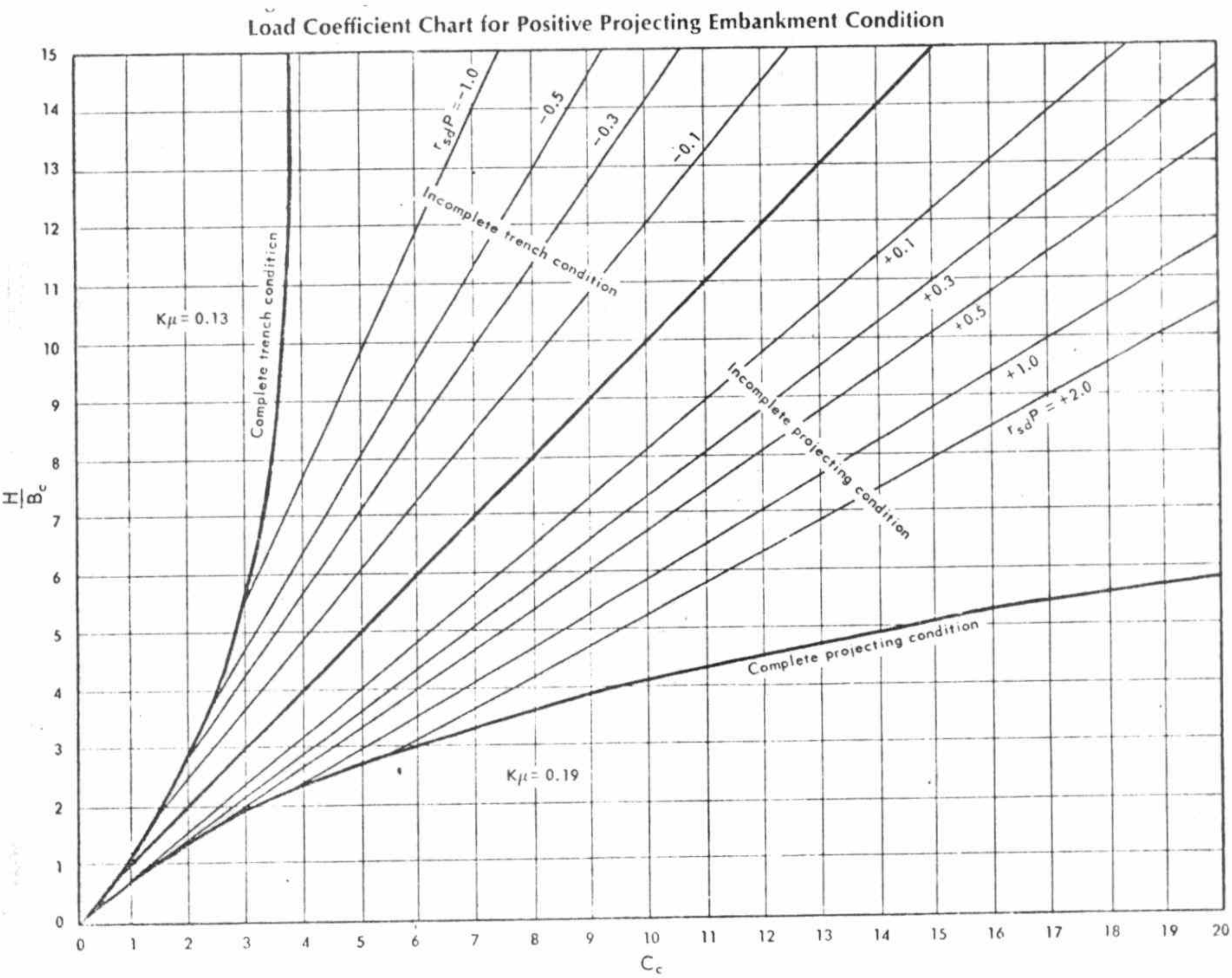
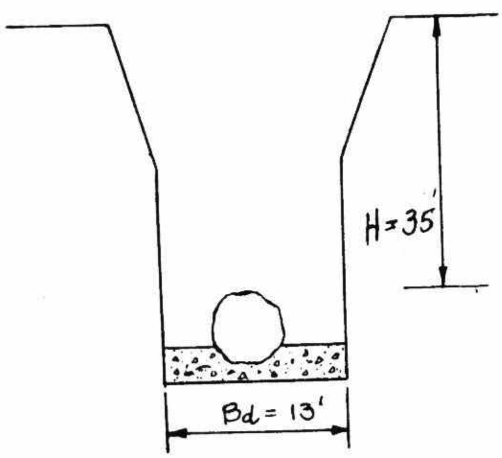

Load Coefficient Chart for Positive Projecting Embankment Condition
$r_{sd}P = -1.0$
-0.5
-0.3
-0.1
Incomplete trench condition
Complete trench condition
Kμ = 0.13
+0.1
+0.3
+0.5
+1.0
$r_{sd}P = +2.0$
Incomplete projecting condition
Complete projecting condition
Kμ = 0.19
$\frac{H}{B_c}$
C_c

Fig. 19

PROBLEM #2. - Positive Projecting Embankment Conduit

GIVEN:



A 48" dia. circular pipe is to be installed in a trench having a width of 13' and a depth of 35' above the top of the pipe. The pipe is to be backfilled with sand and gravel having a unit weight of 110 pcf.

REQUIRED: Class of Pipe.

SOLUTION: assume class B bedding.

From the previous example, it was determined that the transition width was 11.5 ft. (from Fig. 15).

Therefore, this case should be analyzed as a positive projecting conduit since the actual width of trench exceeds the transition width.

Load on Pipe = $Wc = CcwBc^2$

From Fig. 19 for $H/_{Bc} = \frac{35}{4.84} = 7.23$

and for $r_{sd}p$ (assumed) = $(+0.5) \times (+0.7) = +0.35$

$Cc = 10.2$ (From Page 38)

$\therefore \; Wc = 10.2 \times 110 \times 4.84^2 = 26150$ p.l.f.

Determine the load factor from the equation

$$Lf = \frac{1.431}{N-xq}$$

for class B bedding and from table VIII (Page 70)

$$N = 0.707$$

since 70% of the pipe is subjected to lateral earth pressure for class B bedding i.e. $m = 0.7$

from Table VIII $x = 0.594$

From equation $q = \frac{mk}{Cc}\left(\frac{H}{Bc} + \frac{m}{2}\right)$

one can determine the ratio of total lateral earth pressure to total vertical load on the pipe.

m= 0.7, k = 0.33 (assumed)

Cc = 10.2 (from above)

$\therefore \quad q = \frac{0.7 \times 0.33}{10.2} \left(7.23 + \frac{0.7}{2}\right) = 0.173$

$$Lf = \frac{1.431}{0.707 - 0.594 \times 0.173} = 2.37$$

for reinforced concrete pipe, the factor of safety is equal to one.

$$\therefore \quad \text{Required D-Load} = \frac{Wc \times F.S.}{L_f \times Dia.}$$

$$= \frac{26150 \times 1.0}{2.37 \times 4.0} = 2760 \text{ plf/ft. of dia.}$$

$\therefore$ A class V pipe (C-76-68) is required (class B bedding)

D load = 3000 plf/ft. of dia. (From Page 5)

Negative Projecting Conduit

This category is illustrated by fig. 1&2 presented earlier in this article and represents by the situation where a conduit is installed in a shallow trench with the top of the conduit being below the original ground surface and then an embankment is backfilled above the original ground surface.

The load on a negative projecting conduit is determined by the formula:

$$Wc = C_n \, w \, Bd^2$$

where Wc = fill load p.l.f.

Cn = load coefficient for negative projecting conduit

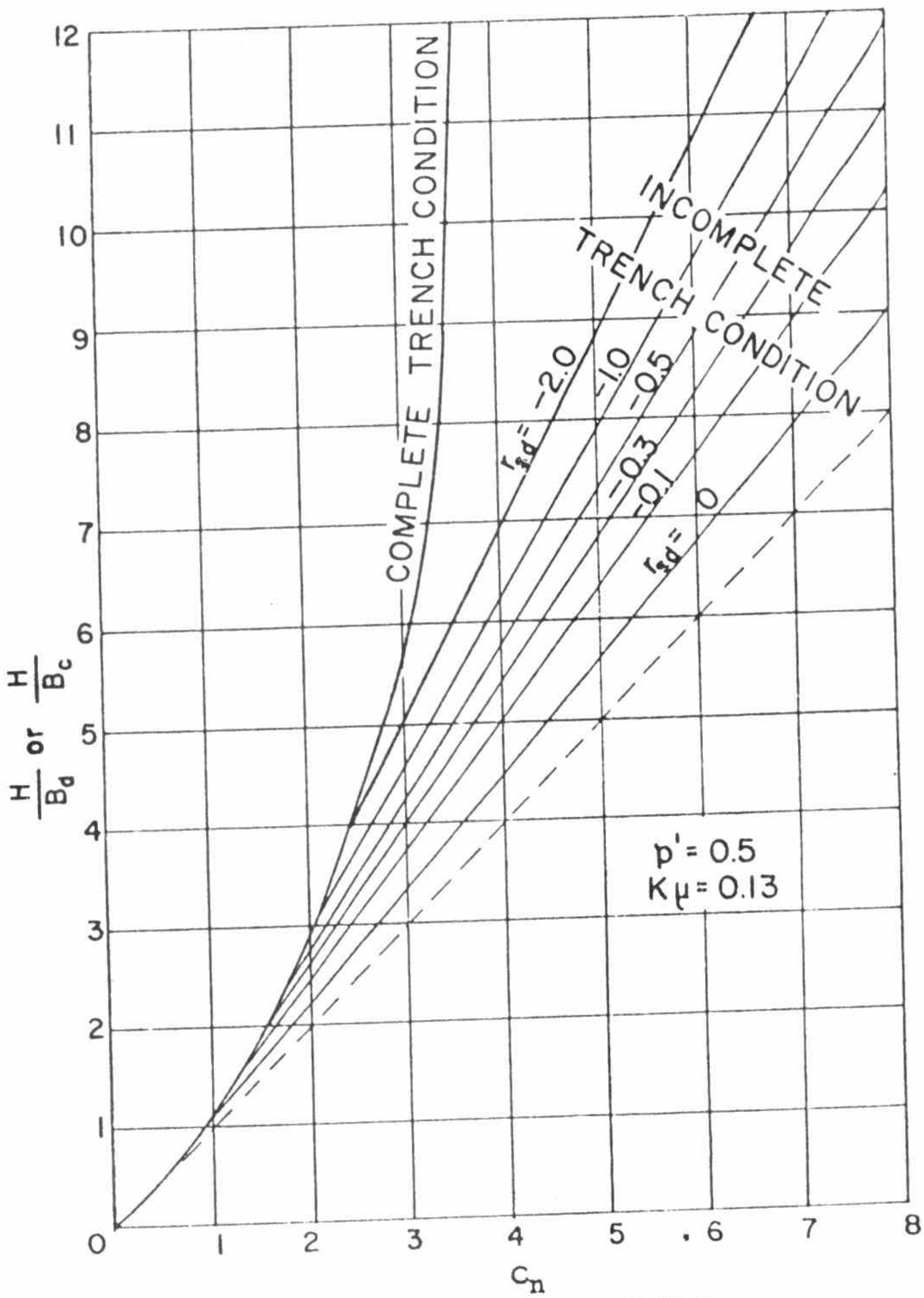
w = unit weight of fill material (p.c.f)

Bd = width of trench (ft).

Load Coefficient, Cn

This factor can be determined from fig. 20, 21, 22, and 23 and is dependent upon the ratio of the height of cover to the width of trench (H/Bc); the value of the projection ratio, (p'); the settlement ratio (rsd); and the product of Rankine's ratio and the coefficient of internal friction ($K\mu$).

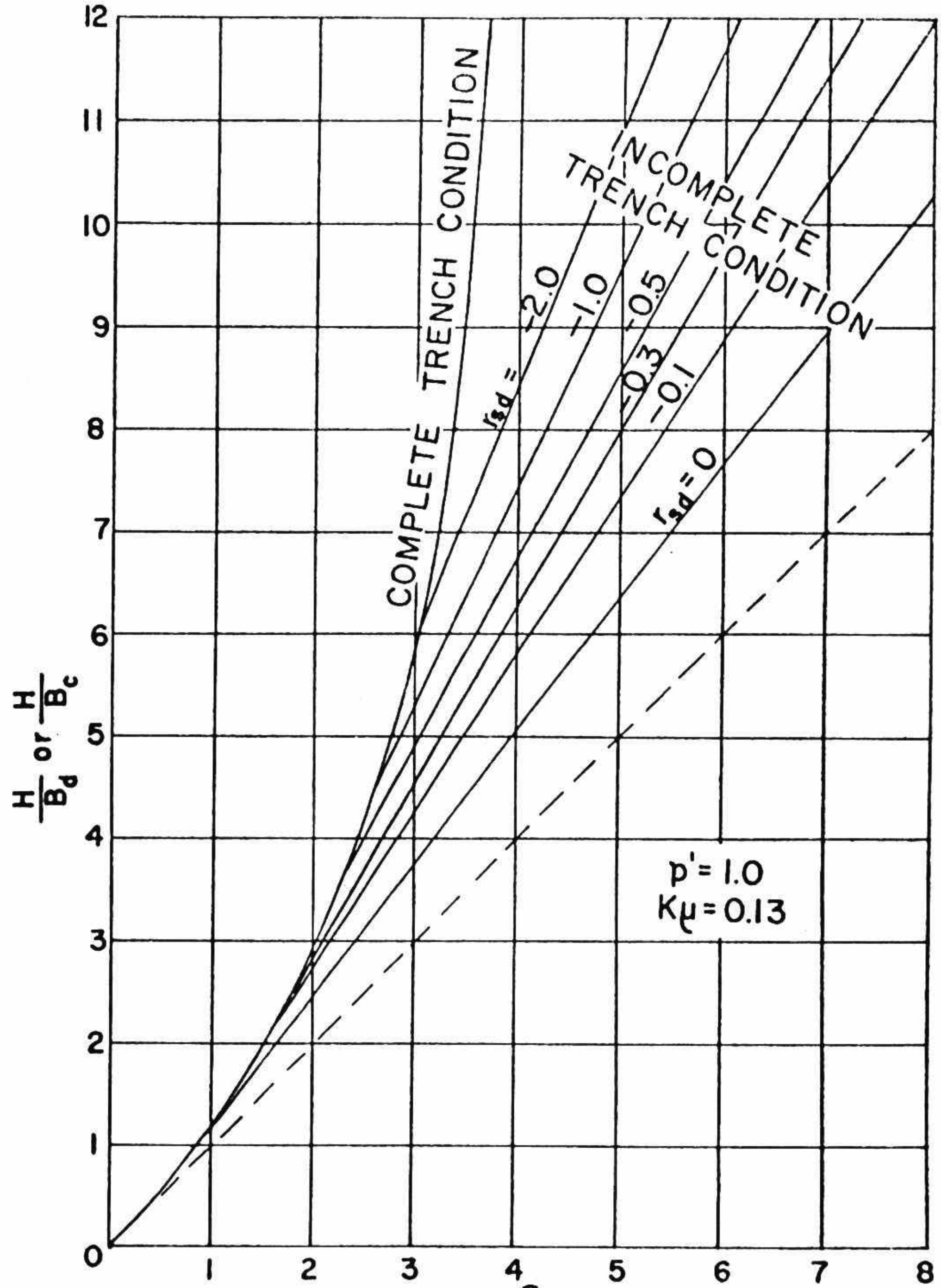
COMPUTATION DIAGRAM FOR SOIL FILL LOADS ON NEGATIVE PROJECTING EMBANKMENT CONDUITS AND IMPERFECT TRENCH CONDUITS



VALUES OF LOAD COEFFICIENT C_n

FIGURE 20

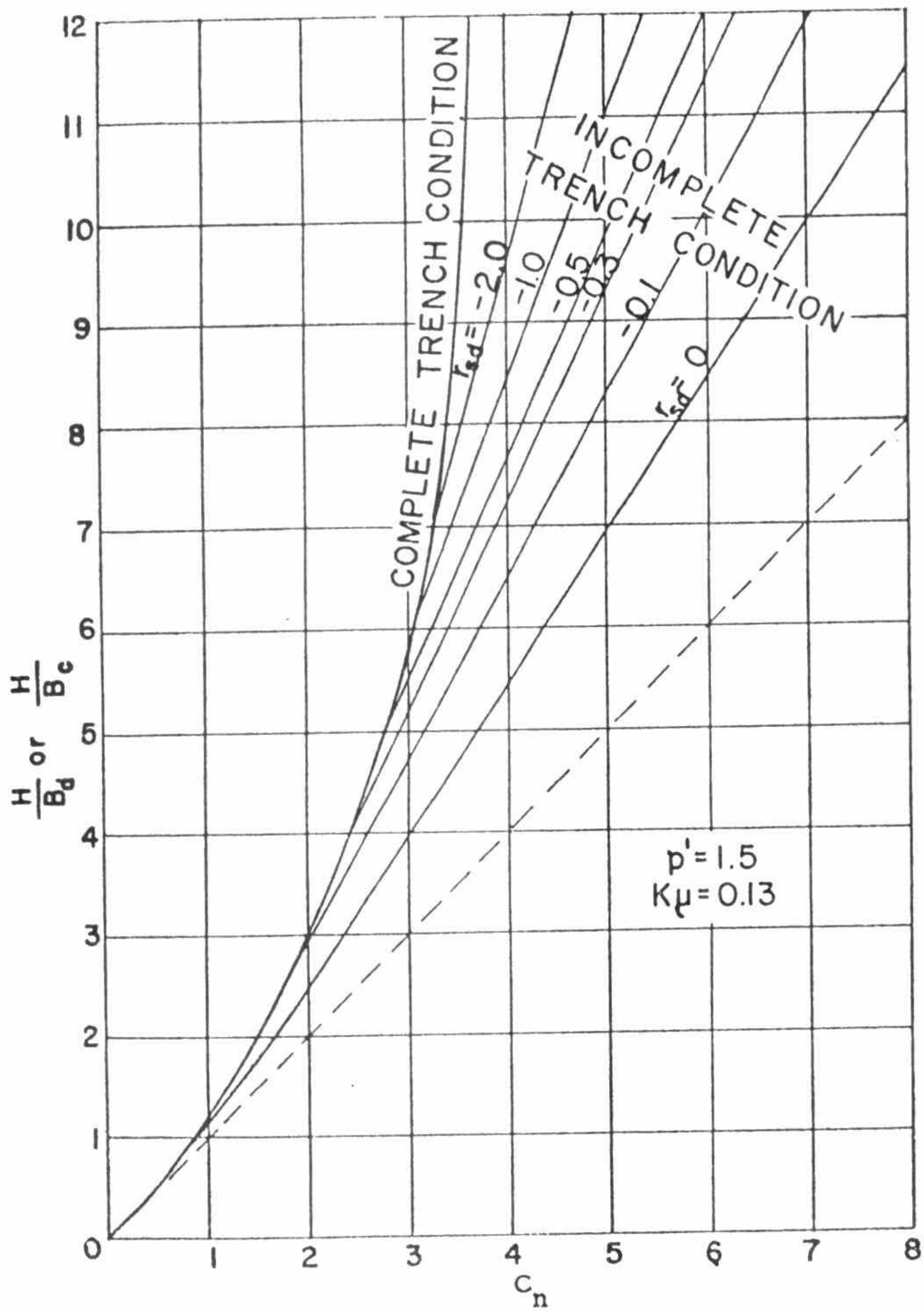
COMPUTATION DIAGRAM FOR SOIL FILL LOADS ON NEGATIVE PROJECTING EMBANKMENT CONDUITS AND IMPERFECT TRENCH CONDUITS



VALUES OF LOAD COEFFICIENT C_n

FIGURE 21

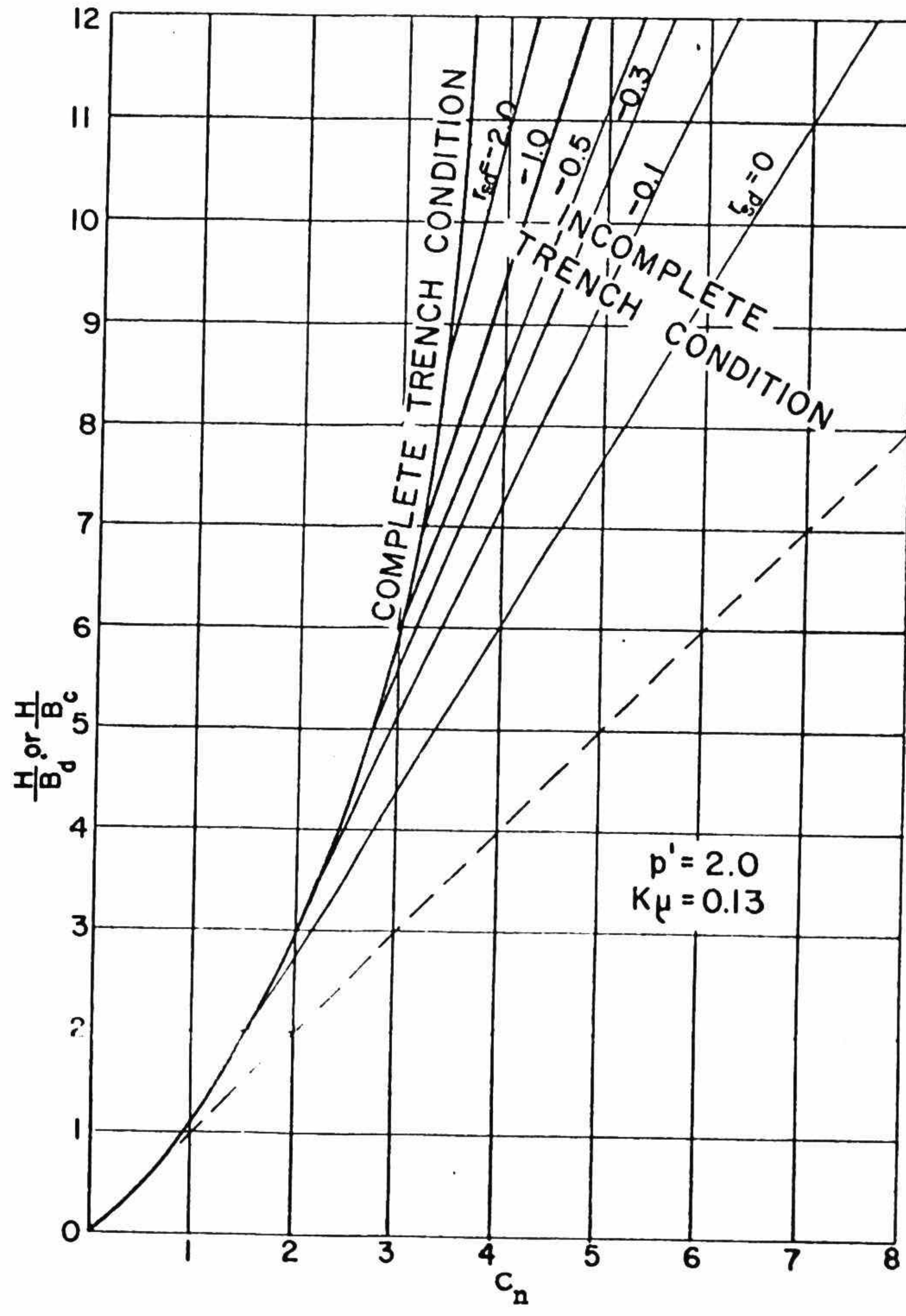
COMPUTATION DIAGRAM FOR SOIL FILL LOADS ON
NEGATIVE PROJECTING EMBANKMENT CONDUITS
AND
IMPERFECT TRENCH CONDUITS



VALUES OF LOAD COEFFICIENT C_n

FIGURE 22

COMPUTATION DIAGRAM FOR SOIL FILL LOADS ON NEGATIVE PROJECTING EMBANKMENT CONDUITS AND IMPERFECT TRENCH CONDUITS



VALUES OF LOAD COEFFICIENT C_n

FIGURE 23

For negative projecting embankment conditions the coefficient of internal friction, μ, is assumed to be 0.2 Therefore the product $K\mu$ = 0.13.

The projection ratio, p', for this type of installation is the distance from the top of the pipe to the surface of the original ground at the time of installation divided by the width of the trench.

The settlement ratio, rsd, can be defined as the ratio of the difference in settlement of the firm ground surface and the critical plane to the compression of soil in the trench.

i.e. $$rsd = \frac{Sg - (Sd + Sf + dc)}{Sd}$$

where rsd = settlement ratio

Sg = settlement of the firm ground surface

Sd = compression of trench backfill over the height p'Bd

Sf = settlement of the bottom of the trench

d_c = deflection of the conduit

The numerical value of the settlement ratio is negative since the interior prism as illustrated in fig. 24 settles more than the exterior prisms. The example problem which follows demonstrates that the load on a conduit is reduced when this type of installation is used.

The range of the value of the settlement ratio, has been determined experimentally, to be between 0 & -2.0. However, it is recommended that a design value of -0.3 be used.

The classification chart (fig. 1) subdivides the negative projecting conduit installation into two sub-categories, complete and incomplete trench condition. Fig. 24 & 25 illustrate the difference in these conditions.

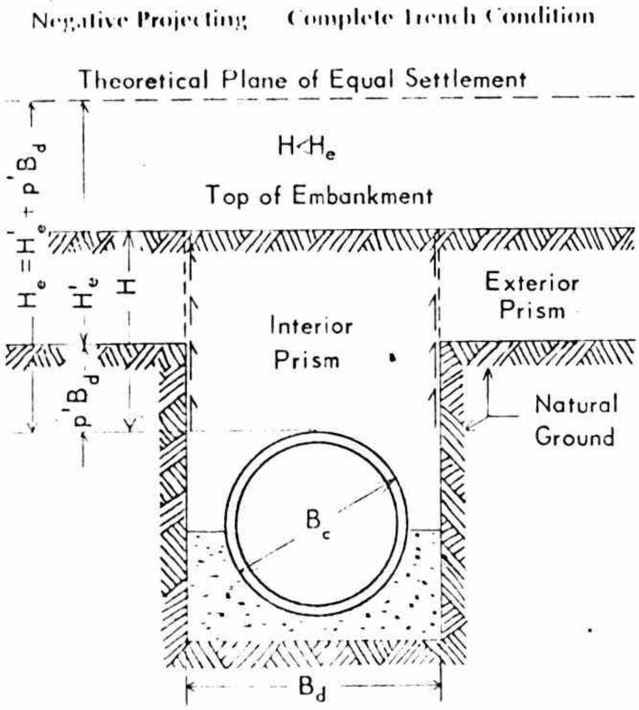


Fig. 24

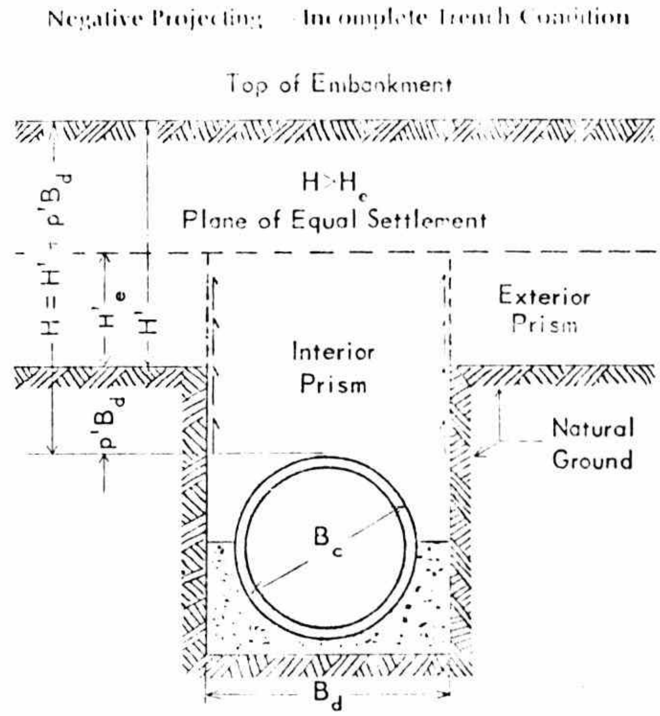


Fig. 25

Induced Trench Conduit

This type of installation utilizes the Marston Theory in reducing the loads acting on a pipe to a value below any other of the previous methods mentioned above. Fig. 26 features the method of construction.

Settlements Which Influence Loads
Induced Trench Installation

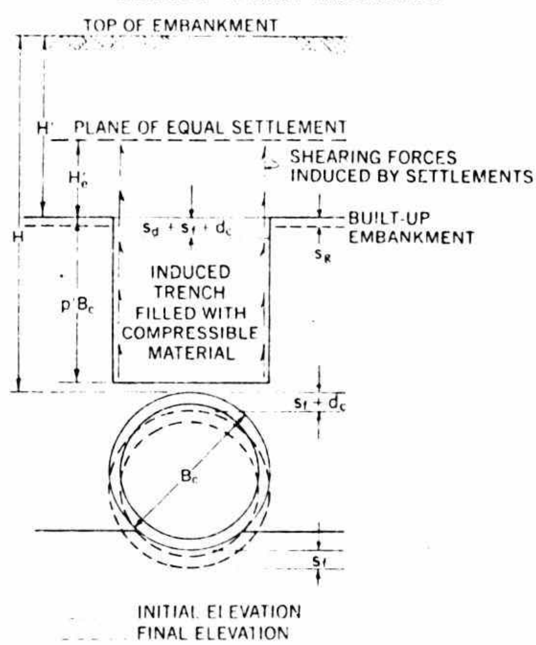


Fig. 26

The pipe is installed in the same manner as a positive projecting conduit and the backfill is compacted for a width equal to twice the outside diameter of the pipe up to an elevation of at least one pipe diameter over the top of the pipe. Next a trench is excavated directly over the pipe (not necessarily right down to the top of the pipe barrel). The width and depth of this trench should be equal to the outside pipe diameter. This trench is then refilled with loose compressable material such as straw, sawdust or organic soil. More than one such induced trench may be constructed in the embankment over the pipe. The balance of the embankment fill is then completed as for a positive projecting conduit (in lifts of 12" usually).

The load on the pipe for the induced trench method is reduced by the upward shearing forces which are generated due to the relative settlement of the interior prism with respect to the adjacent fill.

The formula, $Wc = Cn\ w\ Bd^2$ is used to determine the magnitude of the load on the pipe and

Wc = fill load p.l.f.

Cn = load coefficient for induced trench condition

w = unit weight of fill material p.c.f.

Bd = width of trench (or outside diameter of pipe) ft.

It is important to remember that the width of trench should be kept to a minimum (equal to Bc).

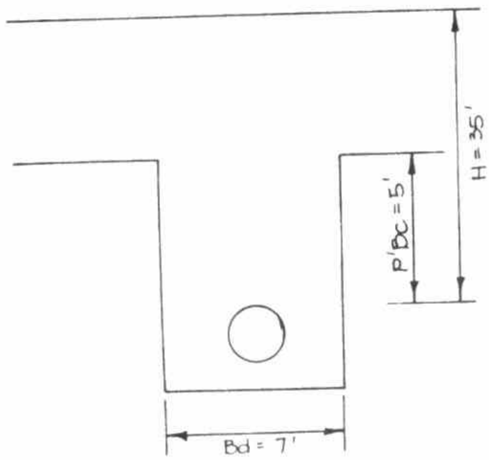
A settlement ratio, equal to - 1.0 is recommended for induced trench conditions.

The load coefficient is determined from the same fig. 20, 21, 22, 23 as for negative projecting embankment conduits. Since the difference in the value of the settlement ratio, the value of the load coefficient will be smaller for the induced trench condition.

This type of construction has also been used with great success in relieving the load anticipated when an existing sewer is to be surcharged with an earth embankment or even a railroad live loading. This avoids the need to replace the pipe with one of superior strength, or to improve the existing bedding.

PROBLEM #3. - Negative Projecting Embankment Conduit

GIVEN:



A 48" diameter concrete pipe is to be installed under a height of fill of 35 feet as illustrated. The pipe is to be placed in a narrow trench having a 7 ft. width and the top of the pipe is about 5 ft. below the level of the original ground. The unit weight of the backfill is 110 pcf.

REQUIRED: To determine the class of pipe required.

SOLUTION: From the definition of the classification of pipe installation earlier in this article, one can see that this problem should be treated as a negative projecting conduit.

The load is determined by the formula,

$$W_c = C_n\ W\ B_d^2$$

for $H/B_d = 35/7 = 5$ and for $r_{sd} = -0.3$ see recommendation page 46

$p'B_c = 5'$ (given)

$\therefore\ p' = 5/4.84 \approx 1.0$

from Fig. 21 $\qquad C_n = 3.3$

$W_c = 3.3 \times 110 \times 7^2 = 17800$ p.l.f.

It is assumed that similar field practices will be taken as per a trench condition installation. This assumes that active lateral earth pressure will not assist in supporting the load on the pipe.

Therefore, the load factor, L_f, will be the same as for class B bedding for a trench condition.

from Fig. 29 $\quad L_f = 1.9$

$\therefore$ Required D-load = $\dfrac{Wc \times F.S.}{L_f \times Dia.}$

$$= \frac{17800 \times 1.0}{1.9 \times 4.0} = 2340 \text{ p.l.f./ft. of dia.}$$

From C-76-68 specifications

Class V pipe has a D-load of 3000 plf/ft. of dia. (From Page 5)

Note that if the lateral earth pressure is assumed to be effective in relieving some of the load on the pipe. (ie if adequate care is taken during construction in bedding and compacting the earth on the sides of the trench, then the load factor, L_f, can be determined from the formula:

$$L_f = \frac{1.431}{N - xq}$$

Where $q = \dfrac{mk}{Cc}\left(\dfrac{H}{Bc} + \dfrac{m}{2}\right)$

$$= \frac{0.7 \times 0.15}{3.3}\left(\frac{35}{4.84} + \frac{0.7}{2}\right)$$

$$= 0.240$$

$N = 0.707$; $x = 0.594$ TABLE VIII

$$L_f = \frac{1.431}{0.707 - 0.594 \times 0.240} = 2.53$$

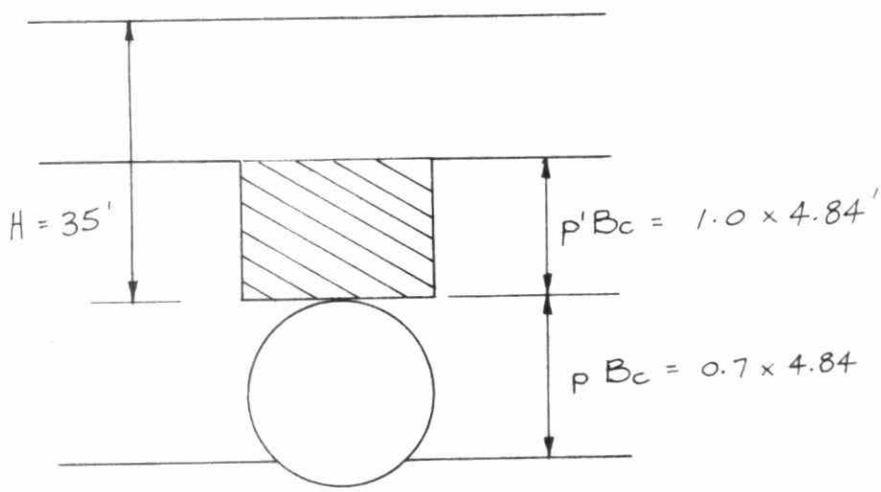
$\therefore$ Required D Load $\dfrac{1.9}{2.53} \times 2340 = 1755$ p.l.f./ft. of dia.

$\therefore$ from C-76-68 specifications a class IV pipe is adequate (D load = 2000 plf/ft. of dia.) (From Page 5)

PROBLEM #4 - Induced Trench Method

Given : A 48" diameter pipe is to be installed by the induced trench method. The height of cover over the top of the pipe is 35 feet. The pipe is to be installed in an embankment consisting of sand and gravel weighing 110 lbs. per cubic foot. The depth of straw in the induced trench is to be assumed to be equal to the outer diameter of the pipe. The width of the induced trench is assumed to be equal to the outer pipe diameter.

Required: Determine class of pipe required.



Solution: Assume Class B bedding

$Bc = 4.84'$ as before

$H/Bc = 35 / 4.84 = 7.23$

from Chart 21 & for rsd = - 1.0

and for $p' = 1.0$ $\therefore$ $rsdp' = -1.0$

$Cn = 3.8$

$\therefore Wc = Cn\ w\ Bd^2$

$= 3.8 \times 110 \times 4.84^2 = 9{,}770$ p.l.f.

The load factor for the induced trench is determined in the same manner as for the positive projecting conduit except that a larger value of q will be applied to account for the settlement of the loose compressible material.

$$Lf = \frac{1.431}{N - x\,q}$$

for Class B bedding & m = 0.7

N = 0.707 , x = 0.594 (From Table VIII)

$$q = \frac{mk}{Cc}\left(\frac{H}{Bc} + \frac{m}{2}\right)$$

$$= \frac{0.7 \times .33}{3.8}\left(7.23 + \frac{.7}{2}\right)$$

$$= 0.461$$

$$Lf = \frac{1.431}{.707 - .594 \times .461} = 3.30$$

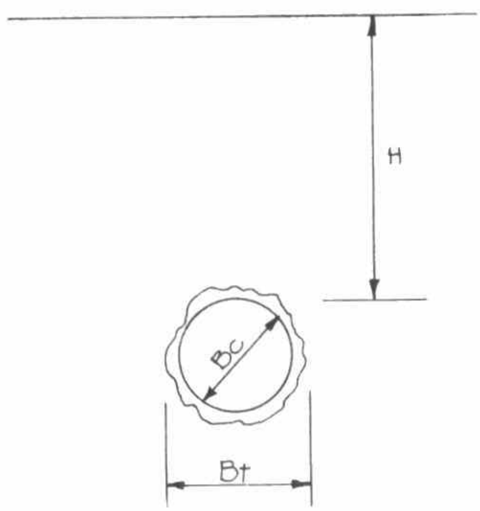
$$\text{Required D load} = \frac{Wc \times F.S.}{Lf \times dia.}$$

$$= \frac{9770 \times 1.0}{3.30 \times 4.0} = 740 \text{ p.l.f./ft.dia.}$$

which from C-76-68 specifications

requires a Class II pipe (1000 p.l.f./ft.of dia.) (From Page 5)

EARTH LOADS ON TUNNEL INSTALLATIONS



When it is necessary for reasons such as excessive depths or conflict with surface works or uses, a tunnel installation may be selected. The load on the pipe may be calculated from the formula:

$$Wt = C_t \; W \; B_t^{\;2} - 2c \; C_t \; B_t$$

Where: W_t = earth load on tunnel (p.l.f.)

W = unit weight of soil (p.c.f.)

B_t = maximum width of tunnel excavation (ft.)

c = coefficient of cohesion (p.s.f.)

C_t = load coefficient for tunnel installation

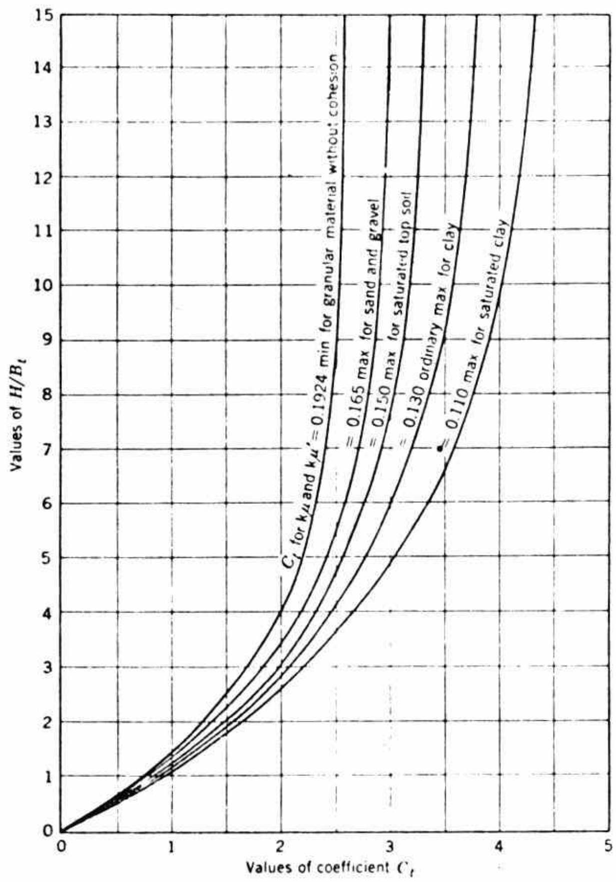
This formula is limited in use to homogeneous soils with even distribution of stresses being subjected to the pipe. The values of the coefficient of cohesion are listed as follows:

clay - very soft	-	40	(p.s.f.)
" medium	-	250	
" hard	-	1000	
sand - loose, dry	-	0	
" silty	-	100	
" dense	-	300	
top soil - saturated	-	100	

If actual soil tests have been taken to determine this value, the amount should be reduced by 33%, to allow for a possible variation in the overall soil body.

The load coefficient is determined from fig. 27 it is interesting to note that these values are the same as the load coefficient for trench conditions.

The designer should be aware that if any voids are left between the tunnel wall and the surrounding earth, then no cohesion will take place and the load on the tunnel will be the same as for the trench conduit installation.



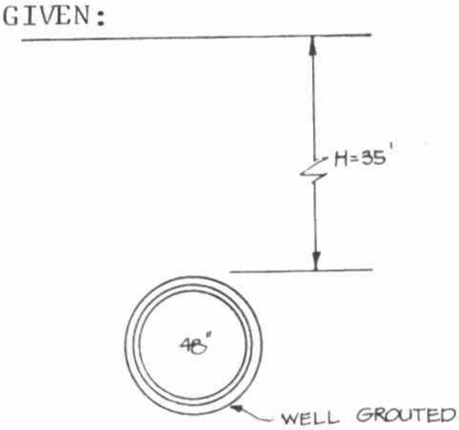
VALUES OF C_t FOR TUNNEL LOADING

Fig. 27

For properly constructed tunnels with excellent contact between the pipe walls and the surrounding earth, a load factor of 3.0 is recommended. Conversely, if the contact area is poor, and voids exist, a factor of 1.9 should be used.

PROBLEM #5 - Tunnel Installation

GIVEN:



A 48" dia. pipe is to be jacked through an ordinary clay (unit weight = 110 pcf) at a depth of 35 ft. over the top of the pipe. Assume the maximum width of excavation is 5 ft.

REUIRED: Determine the class of pipe required.

SOLUTION: From Fig. 27 for $H/B_t = \frac{35}{5} = 7.0$

and for ordinary clay $C_t = 3.3$

from table for values of cohesion, $c = 100$ (psf)

(to be conservative)

$$\therefore W_t = Ct\ W\ B_t^2 - 2c\ C_t\ B_t$$

$$= 3.3 \times 110 \times 5^2 - 2 \times 100 \times 3.3 \times 5$$

$$= 5780 \text{ p.l.f.}$$

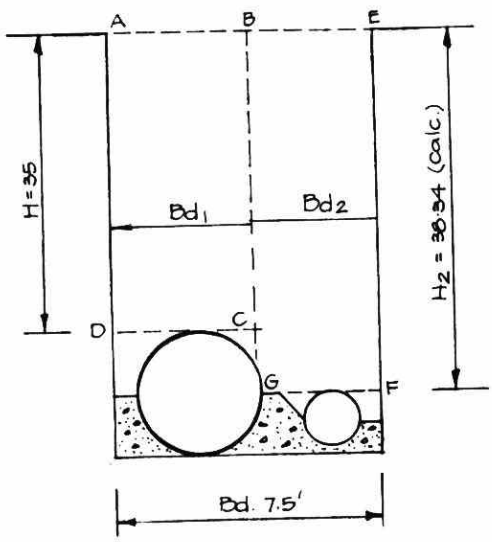
Assuming that the voids between the earth and the pipe will be grouted after the jacking operations have been completed, a load factor of 3.0 can be conservatively estimated.

$$\therefore \text{Required D load} = \frac{Wc \times F.S.}{L_f \times Dia.}$$

$$= \frac{5780 \times 1.0}{3.0 \times 4.0} = 481 \text{ plf/ft. of dia.}$$

From C76-68 specifications, a class II pipe would be required.

Problem # 6 Multiple Installation



A 48" dia. storm sewer is to be installed in the same trench as a 12" dia. sanitary sewer. Inverts are to be level, height of cover over the 48" dia. pipe is 35 ft. Both pipes are to be bedded in 1500 p.s.i. concrete to springline. The backfill material weighs 120 p.c.f. (ordinary clay)

Find: The class of each pipe required.

Solution:

Analyze the 12" dia. pipe first. The barrel of the 48" dia. pipe is sufficiently rigid to assume that the width of trench for the 12" dia. pipe is equal to Bd_2. The load on the pipe from the Marston formula is $Wc = Cd\ W\ Bd_2^2$

for $H/Bd = \dfrac{35 + 4.42 - 1.08}{1.17 + 2 \times 0.5} = \dfrac{38.34}{2.17} = 17.65$

From figure 14 for ordinary clay

$Cd = 3.85$

$Wc = 3.85 \times 120 \times 2.17^2 = 2170$ p.l.f.

From the illustration above, one can observe that prism BEFG will settle more than the adjacent prism ABCD.

The magnitude of the shearing forces can be determined as one-half of the difference of the weight of the prism BEFG and the load calculated according to the Marston formula.

Wt. of Prism BEFG = 38.34 x 2.17 x 1 x 120 = 9,990 p.l.f. The upward frictional force on each side of the Prism BEFG is

$$\frac{9{,}990 - 2170}{2} = 3910 \text{ p.l.f.}$$

This force also acts downward on the face of the adjacent Prism ABCD.

Analyzing the 48" dia. pipe; $H/Bd_1 = 35/5.33 = 4.67$ (where $Bd_1 = Bd - Bd_2$)

from fig.14 $\quad$ Cd = 2.70

$Wc = 2.70 \times 120 \times 5.33^2 = 9180$ p.l.f.

Wt of Prism ABCD = 35 x 5.33 x 1 x 120 = 22,300 p.l.f.

$\therefore$ Upward shearing forces on planes AB & BC = $\frac{22300 - 9180}{2} = 6560$ p.l.f.

Therefore, the load acting on the 48" dia. pipe is the dead load less the upward shearing forces plus the downward shearing forces

or Wc = 22300 - 6560 + 3910 = 19650 p.l.f.

For the 12" dia. pipe

Required D load = $\frac{Wc \times F.S.}{L_f \times \text{dia.}} = \frac{2170 \times 1.5}{2.1 \times 1.0} = 1550$ p.l.f./ft of dia.

Require ASTM C-14 (-70 - Class 1)

(1800 p.l.f./ft. of dia.)

For the 48" dia. pipe

Required D load = $\frac{Wc \times F.S.}{L_f \times \text{dia.}} = \frac{19650 \times 1.0}{2.1 \times 4.0} = 2340$ p.l.f./ft. of dia.

require class V ASTM C-76-68

(3000 p.l.f./ft. of dia.)

Note: The load factor is assumed to be somewhat better than Class B but less than the value of Class A (cradle)

Calculation of Live Loads on Buried Conduits

So far in this article, the attention has been focused on the determination of dead loads. If a buried conduit has been installed with only a shallow depth of cover, say up to 4 feet, then the magnitude of the effect of any applied live load should also be considered.

Live load can be defined as either heavy truck traffic or railroad loading. Airport live loading or railroad loading will not be discussed due to the lack of time.

Highway Loading

There are two types of pavement to be considered in the analysis of highway loading - flexible (asphaltic or gravel) and rigid (concrete). Naturally, as would be expected, the distribution of applied loads differs for the two types. This article will only discuss the effect of applied loads on buried conduits under flexible pavements.

Allowable axle loads vary, with the limit of 18,000 to 20,000 lb. per single axle being common. Therefore, the single wheel loading with dual tires can be taken to be 10,000 lb. This loading is more frequently designated as H-20 loading.

Normal truck tire pressure is 80 p.s.i.

The total live load transmitted to an underground conduit can be determined by calculating the volume of the pressure intensity diagram illustrated in figure 28 below. This volume is closely approximated by an elliptical cylinder and ellipsoid.

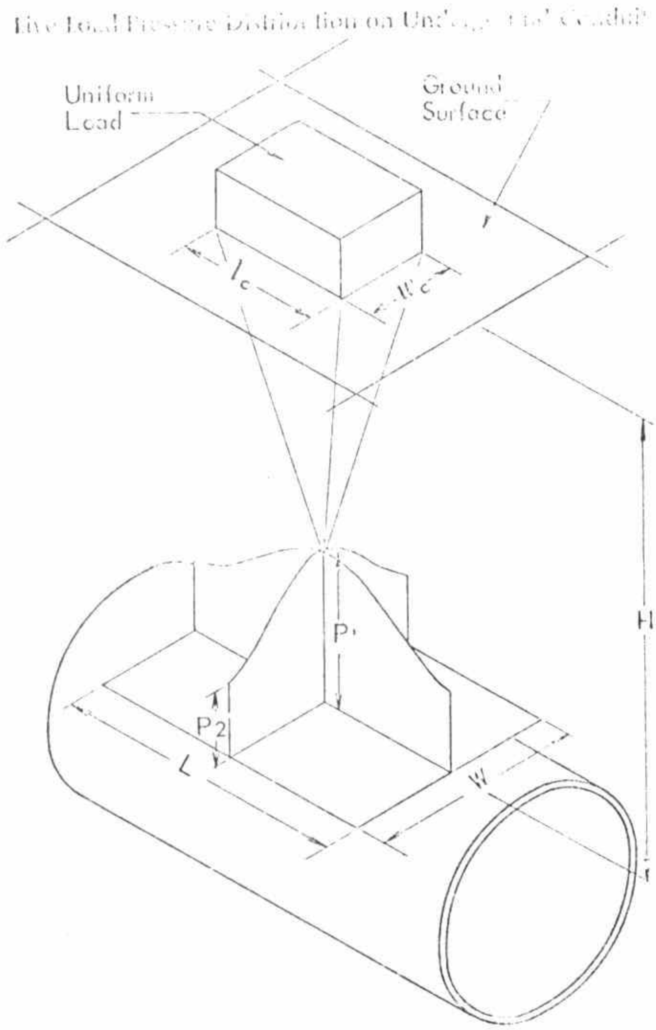


Fig. 28

With reference to figure 28 , the total live load per lineal foot of pipe is determined by the equation

$W_L = \dfrac{\pi W L\ (2p_1 + p_2)}{L + 24}$ where W_L = total live load transmitted to the conduit (p.l.f.)

π = 3.1416

W = width of loaded area (in)

L = length of loaded area (in)

P_1 = vertical unit pressure at the centre of the conduit (p.s.i.)

P_2 = vertical unit pressure at the outside dia. of the conduit (p.s.i)

The values of the vertical unit pressures, p_1 and p_2, can be calculated by the Boussinesq theory which states that the application of a concentrated vertical load to the horizontal surface of any solid body produces a set of vertical stresses on every horizontal plane within the body.

Figure 29 shows the distribution of live load for dual tire truck loading.

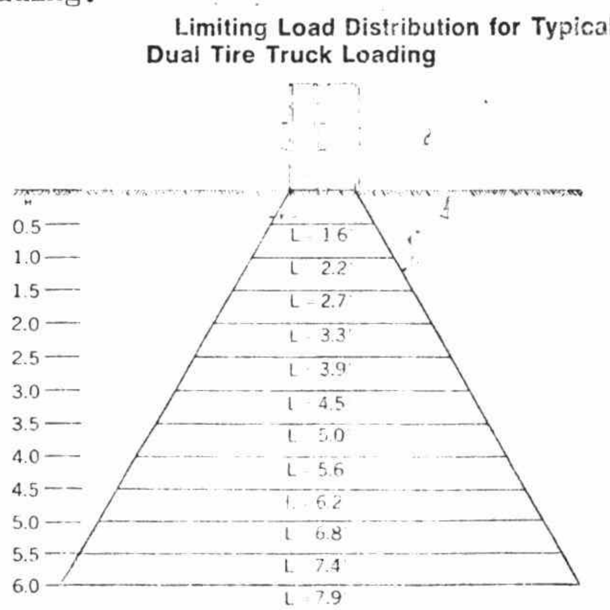


Fig. 29

For this type of loading, the formula $Ac = \frac{P}{P_o}$ is used to determine the relationship of the contact area, Ac (in.^{2}), between the tire and the ground surface, the total load carried by a single wheel, P (lb) and the tire pressure (p.s.i.), P_o .

The contact area is approximately rectangular in shape with the length about 40% greater than the width.

Therefore: $lc = \sqrt{\frac{1.4P}{P_o}}$ & $W_c = \sqrt{\frac{P}{1.4\ P_o}}$

where lc & Wc are the length and width of the tire contact area (inches) respectively.

The effective area over which the distributed load acts on a buried conduit is limited to the outside pipe barrel, B_c and the length is determined from the distribution diagram (fig. 29) This can be equated as $L = 1.0 + \frac{2}{\sqrt{3}}\ H$

where L = length of loaded area (ft)

H = height of fill above top of pipe (ft)

Impact Factor

The following table lists the recommended range of impact factors for depths of cover less than 3 ft.

Impact Factors for Traffic Loads

Height of Cover	Impact Factor
0' - 1'-0	30%
1'-1" - 2'-0"	20%
2'-1" - 2'-11"	10%
3'-0" - and over	0

The table below lists the total live load for various pipe sizes and depth of cover. This is a tabulation of the results of solving the equations $W_L = \frac{\pi\ WL\ (2p_1 + p2)}{L + 24}$

It should be noted that the values of p_1 and p_2 must be determined by the solution of the Boussinesq equation which is presented in soil texts.

Effect of more than one wheel

Two passing trucks may cause twice the distributed live load to be transmitted to the buried conduit as illustrated below:

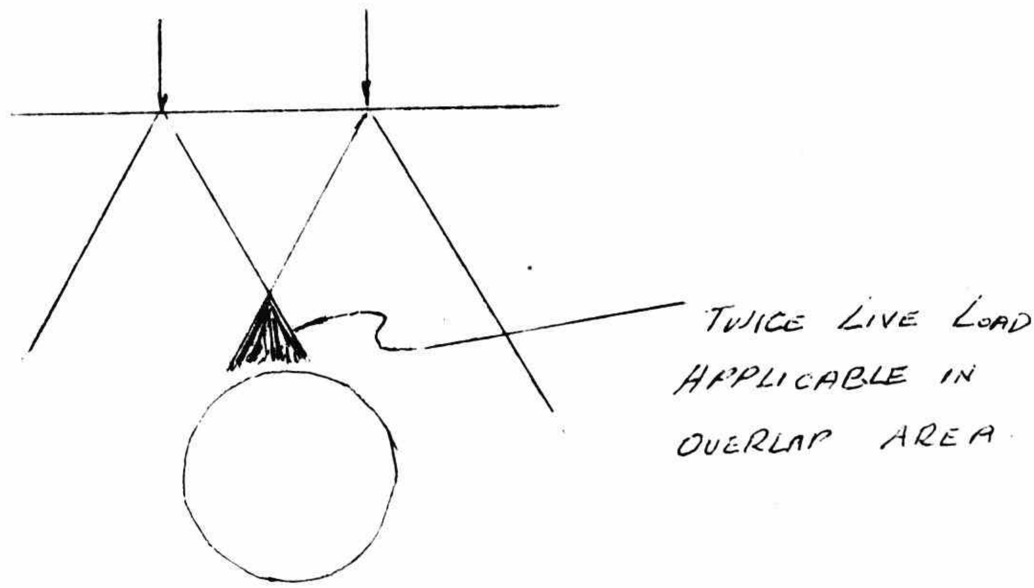


Table VII below is a tabulation of the live load on a conduit for various pipe sizes and depths. (Due to single wheel)

Total Live Load in Pounds Per Lineal Foot

DATA: 10,000 Pound Wheel Load Dual Tires 80 p.s.i. tire pressure Impact not included

D / B_c / H	12" (16)	15" (19.5)	18" (23)	24" (30)	30" (37)	36" (44)	42" (51)	48" (58)	60" (72)	72" (86)	84" (100)	96" (114)
0.5	3730	4550	4550	4550	4550	4550	4550	4550	4550	4550	4550	4550
1.0	2100	2555	3020	3460	3460	3460	3460	3460	3460	3460	3460	3460
1.5	1270	1560	1830	2380	2580	2580	2580	2580	2580	2580	2580	2580
2.0	795	969	1141	1490	1837	1875	1875	1875	1875	1875	1875	1875
2.5	522	636	750	979	1207	1436	1526	1526	1526	1526	1526	1526
3.0	361	441	520	679	837	996	1154	1221	1221	1221	1221	1221
3.5	316	385	454	592	731	869	1007	1146	1185	1185	1185	1185
4.0	237	289	341	444	548	652	755	859	998	998	998	998

Interpolate for intermediate pipe sizes and or fill heights.

TABLE VII

Problem # 7 - Live Load

A 48" dia. sewer is to be installed under an asphaltic road surface at a depth of cover of 2'-0" in a trench condition (width of excavated trench = 7 ft.) Assume Class B bedding and granular backfill weighing 110 p.c.f.

Find: The class of pipe required.

Solution: From Table VII (page 63) W_L = 1875 p.l.f.

For H=2' Impact Factor = I_f = 1.20

∴ Effective total live load = $W_L \times I_f$ = 1875 × 1.20 = 2250 p.l.f.

Effective dead load can be calculated from the formula

$Wd = Cd\ w\ Bd^2$

for $H/B_d = 2/7 = 0.29$

From fig. 14 (page 29) Cd = 0.27

$Wd = 0.27 \times 110 \times 7^2 = 1456$ p.l.f.

Total load on pipe = $Wd + W_{L_T}$ = 1456 + 2250 = 3706 p.l.f.

for Class B bedding L_f = 1.9

$$\text{Required D-load} = \frac{\left(Wd + W_{L_T}\right) \times F.S.}{L_f \times Dia.}$$

$$= \frac{3706 \times 1.0}{1.9 \times 4.0} = 487 \text{ p.l.f.}$$

which from C-76-68 specifications

requires a Class II pipe (1000 p.l.f./ft.of dia.)

LOAD FACTOR

The in-place supporting strength of a rigid conduit depends upon the inherent strength of the pipe, the type of the foundation beneath the pipe and the degree of compaction of the fill material adjacent to the pipe.

The inherent strength of the pipe can be taken as the resistance to the 3-edge bearing test which gives the most severe loading to which the pipe will be subjected.

The designer's task is one of selecting the most economical combination of pipe strength and pipe bedding classes.

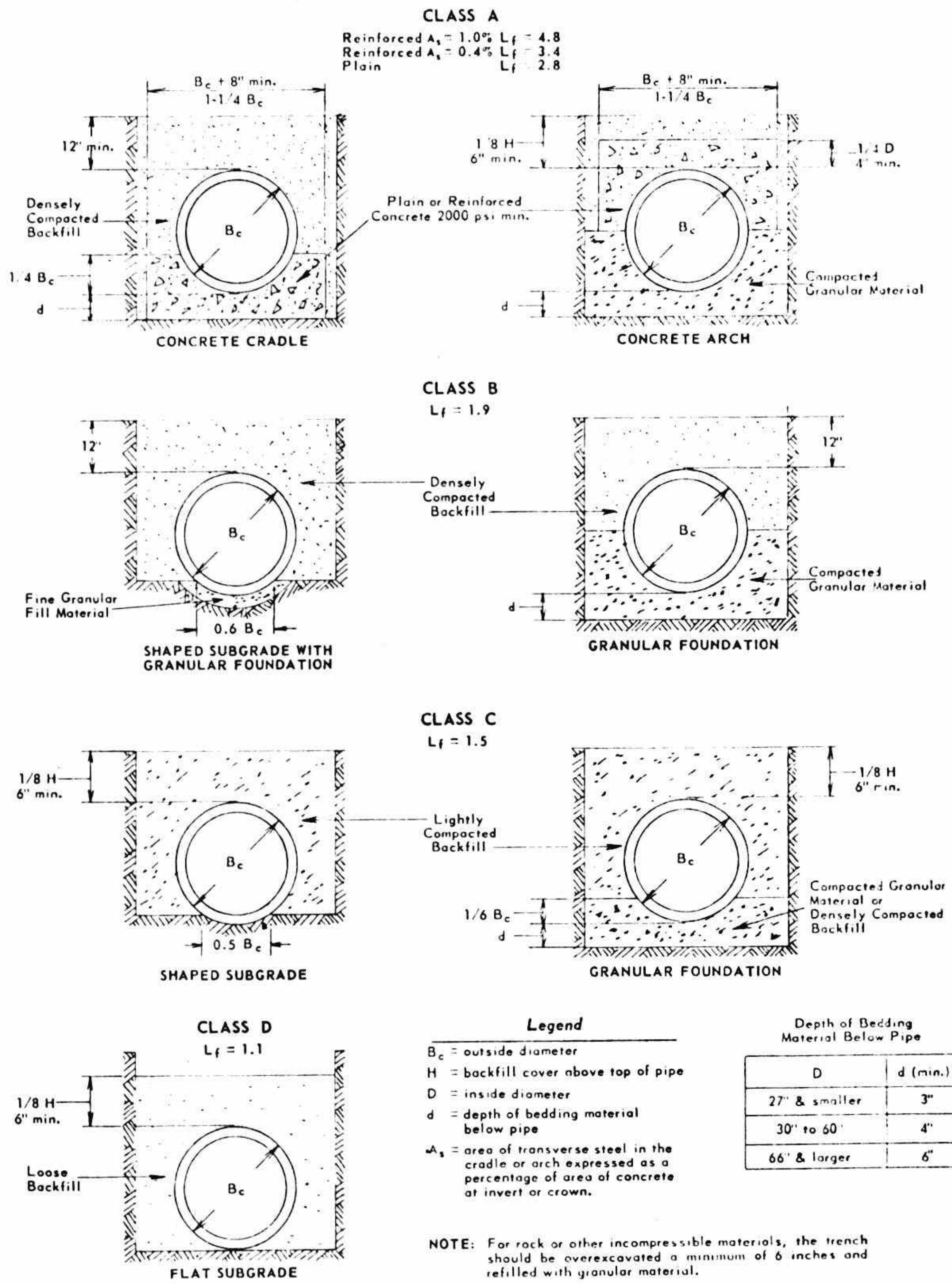
To relate this inherent strength to the in-place supporting strength a load factor is applied. This factor considers how the distribution of total vertical load varies with quality of the contact between the bedding material and the pipe and is the value by which the manufactured strength can be increased to account for the actual supporting strength encountered in field conditions.

The load factor is also dependent upon the magnitude of the lateral earth pressure and the area of the pipe to which this pressure is subjected.

In other words, the load factor is the field supporting strength divided by the strength determined by the 3-edge bearing test. Load factors have been determined experimentally for typical trench and embankment conditions.

A. Trench Conduit - For trench conduit installations there are four classes of bedding commonly used. These are described in figure 29

Trench Beddings



Legend

- B_c = outside diameter
- H = backfill cover above top of pipe
- D = inside diameter
- d = depth of bedding material below pipe
- A_s = area of transverse steel in the cradle or arch expressed as a percentage of area of concrete at invert or crown.

Depth of Bedding Material Below Pipe

D	d (min.)
27" & smaller	3"
30" to 60"	4"
66" & larger	6"

NOTE: For rock or other incompressible materials, the trench should be overexcavated a minimum of 6 inches and refilled with granular material.

Fig. 29

B. Encasement

Where the required safe supporting strength cannot be obtained by the use of one of the above-noted bedding selections, the pipe can be encased in concrete. Figure 30 shows the increase in supporting strength (plf) due to various thicknesses of encasement. For a compressive strength of concrete other than 3000 psi the values should be proportioned accordingly.

To illustrate the use of the chart the following example is given.

Assuming a 24" diameter concrete pipe is to be encased with 6" of 2000 psi concrete. From Figure 30 one can see that the increase in supporting strength is 12500 plf for 3000 psi concrete Professor Spangler, in answer to a questionnaire on the use of this chart, advised that the values of increase in supporting strength should be reduced by 7/10 in order to equate the value to the increase in 3-edge bearing strength.

Therefore, for this particular assumed example the increase in supporting strength = $12{,}500 \times 7/10 \times \frac{2000}{3000} = 5830$ plf

Assuming type B bedding, the load factor is 1.9 (see figure 14 . Also assume class III pipe meeting ASTM specifications C-76-68. Therefore, the 3-edge bearing strength is 2700 plf (see TABLE I). Therefore, the safe supporting strength = load factor x 3-edge bearing strength.

or Safe supporting strength = 1.9 (2700 + 5830)

= 16,200 plf.

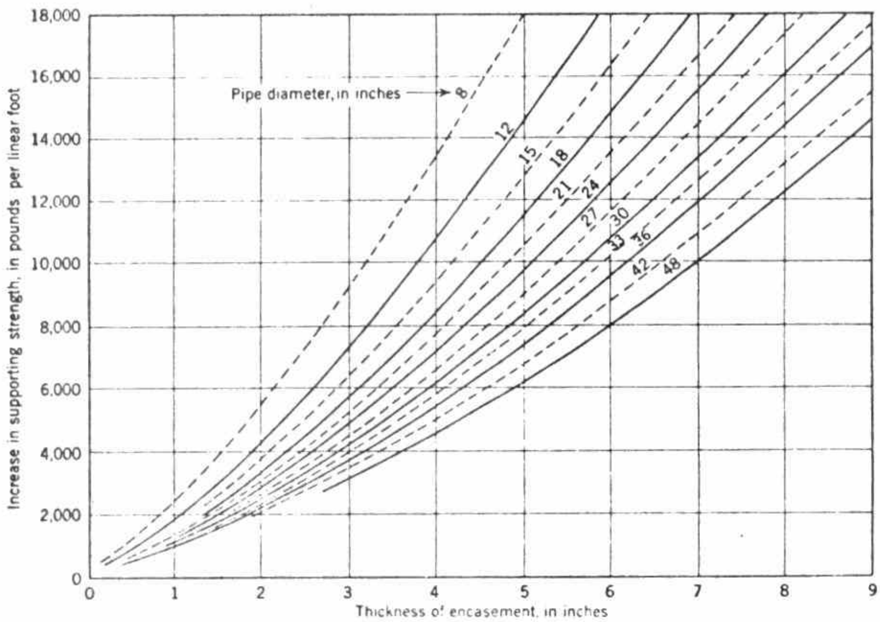


Fig. 75. Effect of 3,000-Lb Concrete Encasement of Various Thicknesses on Supporting Strength of Pipe Under Trench-Laying Conditions

Fig. 30

Note that on analyzing this same pipe for <u>unencased</u> conditions the safe supporting strength = 1.9 x 2700 = 5130 plf. Encasement in this case represents an increase in safe supporting strength of $\frac{16200 - 5130}{5130}$ 100 = 216%

C. <u>Embankment Conduits</u>

The active soil pressure against the sides of the pipe aids in the support of the applied loads. Like the trench condition, the load factor for embankment conduits depends upon the class of bedding. In addition the magnitude of the active lateral earth pressure and the area of the pipe over which this pressure acts also influence the value of the load factor.

The load factor for projecting embankment conduits may be calculated by the formula:

$$L_f = \frac{1.431}{N-xq}$$

Where L_f is the load factor.

N is a parameter dependent upon type of bedding.

x is a parameter dependent upon the area over which the active lateral earth pressure acts.

q is the ratio of total lateral pressure to total vertical load on the pipe.

D. <u>Positive Projecting Conduits</u>

$$q = \frac{mk}{Cc}\left(\frac{H}{Bc} + \frac{m}{2}\right)$$

Where m is the fraction of the pipe subjected to lateral pressure.

k is Rankine's ratio (ie $\frac{\text{unit lateral pressure}}{\text{unit vertical pressure}}$)

This value is assumed to be equal to 0.33 from experience.

Values of N, m, and x are given in the table below.

Table VIII

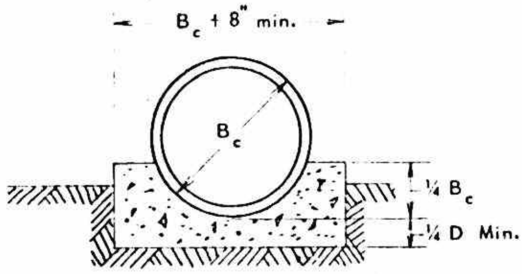
Class of Bedding	N	m	x	x^{1*}
A. Reinf. cradle	0.421 to 0.505	0.0	0.000	0.150
Unreinf. cradle	0.505 to 0.636	0.5	0.423	0.856
B.	0.707	0.7	0.594	0.811
C.	0.840	0.9	0.655	0.678
D.	1.310	1.0	0.638	0.638

*Note: values of x^1 are to be used for Class A bedding only.

Figure 31 shows the classification of embankment-beddings.

Embankment Beddings

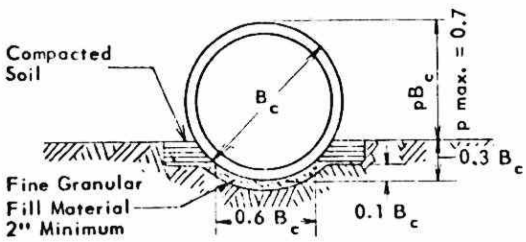
CLASS A



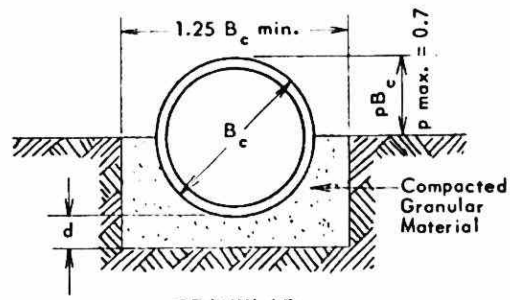
CONCRETE CRADLE

ROCK OR OTHER NONCOMPRESSIBLE FOUNDATION – Where ledge rock, rocky or gravelly soil, hard pan or other unyielding foundation material is encountered, the hard unyielding material shall be excavated below the elevation of the concrete cradle (Class A) or the bottom of the pipe or pipe bell (Class B & C Beddings) for a depth of at least 6 inches or ½ inch for each foot of fill over the top of the pipe, whichever is greater, but not more than ¾ the nominal diameter of the pipe. For Class D Bedding, the depth shall be 6 inches. The width of the excavation shall be one foot greater than the outside diameter of the pipe. The excavation shall be refilled with selected fine compressible material, such as silty clay or loam, lightly compacted and shaped as required for the specified class of bedding.

CLASS B

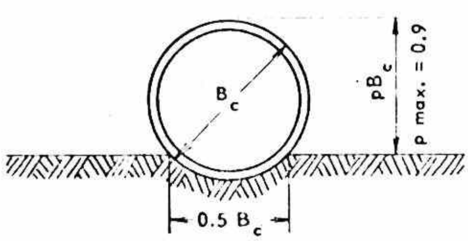


SHAPED SUBGRADE WITH GRANULAR FOUNDATION

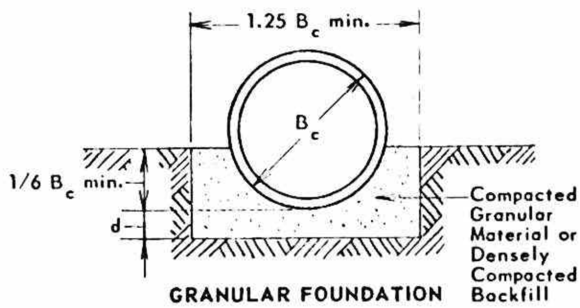


GRANULAR FOUNDATION

CLASS C

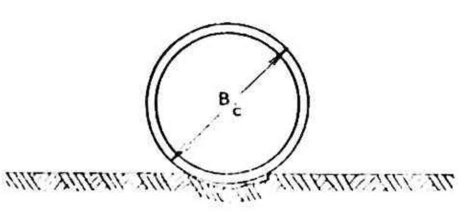


SHAPED SUBGRADE



GRANULAR FOUNDATION

CLASS D



FLAT SUBGRADE

Legend

B_c = outside diameter
H = backfill cover above top of pipe
D = inside diameter
d = depth of bedding material below pipe

Depth of Bedding Material Below Pipe

D	d (min.)
27" & smaller	3"
30" to 60"	4"
66" & larger	6"

Fig. 31

Load factors for positive projecting conduits range from 11.3 to 1.1 for class A to class D beddings respectively.

E) Negative Projecting Conduits

The load factor may be either the same as for trench conditions or the value calculated from the equation:

$Lf = \frac{1.431}{N-xq}$. The choice of method is dependent upon the degree of construction effort made to ensure that the active lateral earth pressure will act on the sides of the pipe.

If some confidence can be relied upon to provide for this support, then the value of lateral pressure can be calculated using the formula, $q = \frac{mk}{Cc}\left(\frac{H}{Bc} + \frac{m}{2}\right)$

Where k is assumed to be 0.15.

For negative projecting conduits the range in load factor is 5.1 to 1.1.

F) Induced Trench Conduits

In this type of installation equations,

$Lf = \frac{1.431}{N-xq}$ & $q = \frac{mk}{Cc}\left(\frac{H}{Bc} + \frac{m}{2}\right)$ can also be used in determining the value of the load factor.

Values of the load factor for induced trench conduits range from 7.1 to 1.7.

It should be remembered that the value, 1.431 in the numerator of the load factor formula is applicable only to circular pipe.

The value of 'm' is not necessarily the same as the projection ratio, 'p'. Only when proper compaction of the bedding and the side fill materials is accomplished, is the relationship $(m = p)$ true. In this situation, the active lateral pressure acts over the entire height of the conduit.

S U M M A R Y

48" @ 35' of cover. Class B bedding throughout.

Type of Installation	Trench	Positive Projecting Embankment	Negative Projecting Embankment	Induced Trench	Tunnel	Live Load **	Multi-Installation	Dead Load on Min. Trench Width
Load on Pipe p.l.f.	12930	26150	17800	9770	5780	3706	19650	22400
Required D Load p.l.f. per ft.dia.	1702	2760	2340 (*1755)	740	481	487	2340	2950 assuming Lf = 1.9
Required Class	IV	V	V *(IV)	II	II	II	V	V

** 48" dia. but only 2.0' of cover.

* When care is taken to ensure compaction is excellent.

TABLE IX

TABLE X

48" BACKFILL LOADS ON CIRCULAR PIPE IN TRENCH INSTALLATION 48"

* 100 POUNDS PER CUBIC FOOT BACKFILL MATERIAL — LOADS IN POUNDS PER LINEAR FOOT

A — SAND AND GRAVEL Kμ'—0.165

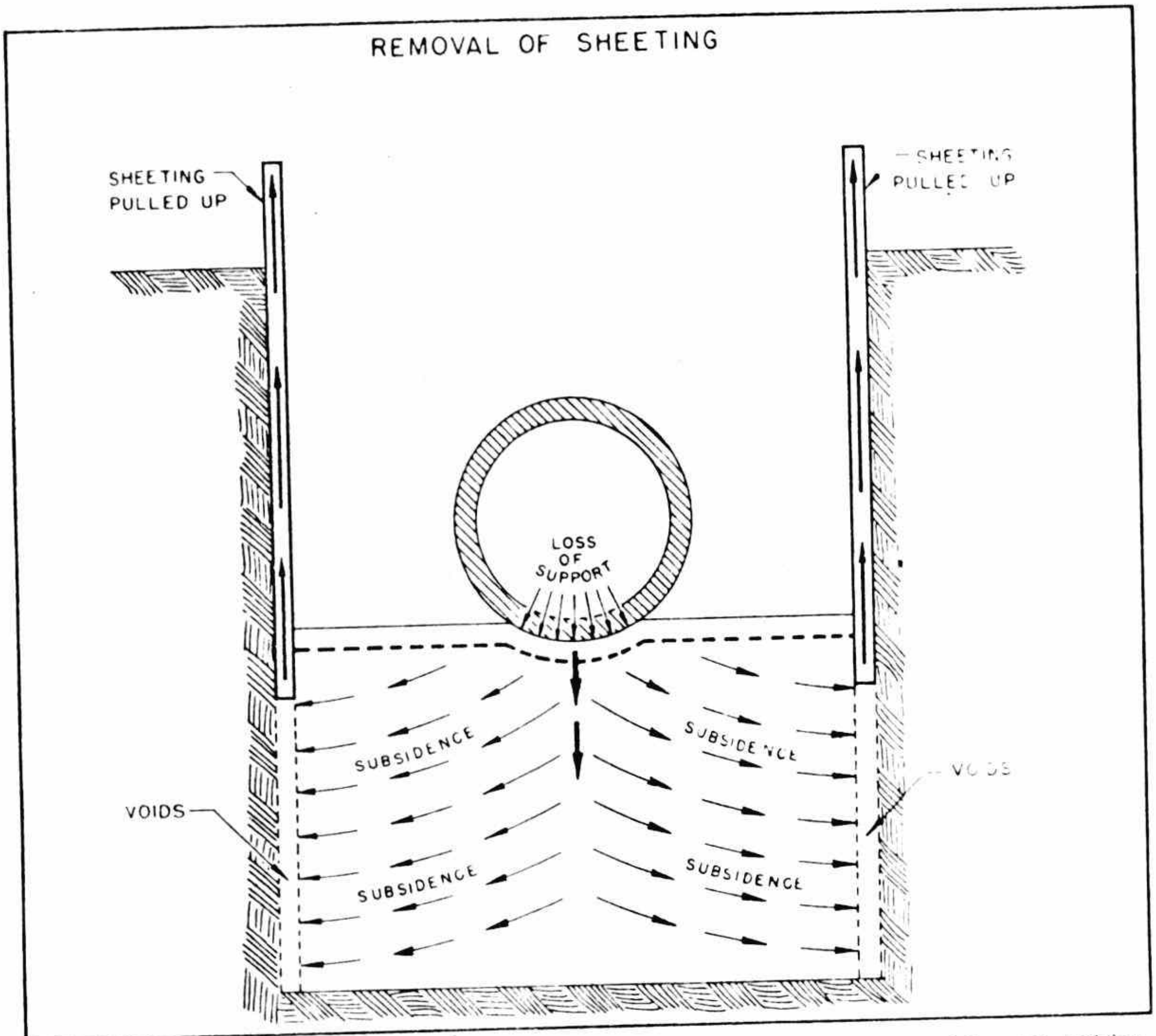
HEIGHT OF BACKFILL H ABOVE TOP OF PIPE, FEET	TRENCH WIDTH AT TOP OF PIPE										▲TRANSITION WIDTH
	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	11'-0"	12'-0"	
5	2870	2950									6'- 8"
6	[illegible]	[illegible]	3691								7'- 1"
7	[illegible]	4173	4490								7'- 6"
8	4273	[illegible]	5057	5354							7'-11"
9	4695	5134	5570	6014	6287						8'- 4"
10	[illegible]	[illegible]	[illegible]	6555	7041						8'- 6"
11	[illegible]	[illegible]	[illegible]	7074	[illegible]	7776					8'- 8"
12	[illegible]	[illegible]	6900	7571	[illegible]	8508					8'- 9"
13	[illegible]	[illegible]	[illegible]	8043	[illegible]	9235					8'-11"
14	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	9966				9'- 1"
15	[illegible]	[illegible]	[illegible]	[illegible]	9664	[illegible]	10690				9'- 2"
16	7120	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	11420				9'- 4"
17	7401	8166	8977	9775	[illegible]	[illegible]	12150				9'- 5"
18	[illegible]	8492	9324	[illegible]	[illegible]	11660	12710	12870			9'- 7"
19	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	12320	13210	13600			9'- 9"
20	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	14330			9'-10"
21	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	14160	15060			9'-11"
22	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	14610	15640	15770		10'- 1"
23	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	13930	15050	16120	16510		10'- 2"
24	[illegible]	[illegible]	[illegible]	12190	13270	14360	15470	16560	17230		10'- 3"
25	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	17960		10'- 5"
26	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	18680		10'- 6"
27	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	16640	17870	19400		10'- 7"
28	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	18270	20120		10'- 8"
29	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	17260	[illegible]	20850		10'-10"
30	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	21590		10'-11"
31	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	16670	[illegible]	19410	22200	22320	11'- 1"
32	[illegible]	[illegible]	[illegible]	[illegible]	15670	16950	[illegible]	19760	[illegible]	23020	11'- 1"
33	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	20100	[illegible]	23740	11'- 3"
34	[illegible]	[illegible]	[illegible]	[illegible]	16040	17490	[illegible]	20440	23440	24490	11'- 4"
35	[illegible]	[illegible]	[illegible]	[illegible]	16270	17740	19240	20760	[illegible]	25220	11'- 5"
36	[illegible]	[illegible]	[illegible]	[illegible]	16480	17990	19520	21060	24210	25930	11'- 6"
37	[illegible]	[illegible]	[illegible]	[illegible]	16690	18220	19780	21360	24580	26680	11'- 8"
38	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	20040	21660	24940	27410	11'- 9"
39	[illegible]	[illegible]	[illegible]	[illegible]	17080	[illegible]	20290	21940	25290	28130	11'-10"
40	[illegible]	[illegible]	[illegible]	15670	17260	[illegible]	[illegible]	22210	25620	28820	11'-11"

B — SATURATED TOP SOIL Kμ'—0.150

TRENCH WIDTH AT TOP OF PIPE										▲TRANSITION WIDTH	HEIGHT OF BACKFILL H ABOVE TOP OF PIPE, FEET
6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	11'-0"	12'-0"		
2902	2950									6'- 7"	5
3406	3691									7'- 0"	6
3888	4233	4490								7'- 4"	7
4348	4740	5134	5354							7'- 9"	8
4787	5227	5668	6110	6287						8'- 2"	9
5206	5693	6181	6671	7041						8'- 5"	10
5606	6139	6674	7210	7776						8'- 6"	11
5989	6567	7147	7730	8315	8508					8'- 8"	12
6354	6976	7602	8231	8862	9235					8'- 9"	13
6702	7369	8039	8713	9389	9966					8'-11"	14
7035	7745	8459	9177	9899	10690					9'- 0"	15
7353	8105	8863	9625	10390	11160	11420				9'- 2"	16
7657	8450	9250	10060	10870	11680	12150				9'- 3"	17
7947	8781	9623	10470	11320	12180	12870				9'- 5"	18
8223	9098	9981	10870	11770	12670	13600				9'- 6"	19
8488	9401	10320	11250	12190	13140	14090	14330			9'- 8"	20
8740	9692	10660	11630	12610	13590	14580	15060			9'- 9"	21
8981	9971	10970	11980	13000	14030	15060	15770			9'-10"	22
9211	10240	11260	12330	13390	14460	15530	16510			9'-11"	23
9431	10490	11570	12660	13760	14870	15980	17110	17230		10'- 1"	24
9641	10740	11850	12980	14120	15260	16420	17590	17960		10'- 2"	25
9841	10970	12120	13290	14460	15650	16850	18050	18680		10'- 3"	26
10030	11200	12380	13580	14800	16020	17260	18500	19400		10'- 4"	27
10220	11410	12630	13870	15120	16380	17660	18940	20120		10'- 5"	28
10390	11620	12870	14140	15430	16730	18040	19370	20850		10'- 7"	29
10560	11820	13100	14410	15730	17070	18420	19780	21590		10'- 8"	30
10720	12010	13320	14660	16020	17390	18780	20180	22320		10'- 9"	31
10870	12190	13540	14910	16300	17710	19130	20570	23020		10'-10"	32
11010	12360	13740	15140	16570	18010	19470	20950	23740		10'-11"	33
11150	12530	13940	15370	15830	18310	19800	21310	24490		11'- 0"	34
11280	12690	14130	15590	17080	18590	20120	21670	24800	25220	11'- 1"	35
11410	12840	14310	15800	17320	18870	20430	22010	25220	25930	11'- 3"	36
11530	12990	14480	16010	17560	19130	20730	22350	25630	26680	11'- 4"	37
11640	13130	14650	16200	17780	19390	21020	22670	26020	27410	11'- 5"	38
11760	13260	14310	16390	18000	19640	21300	22990	26410	28130	11'- 6"	39
11860	13390	14960	16570	18210	19880	21580	23290	26780	28820	11'- 7"	40

of the earth prism over the entire trench width. The trench width to be used when timber is removed is the full outside limits of the excavation.

Figure 32 illustrates the settlement of a pipe or the pipe bedding material when timber is removed.



Removal of sheeting which extends below the invert of the pipe causes subsidence of the pipe bedding.

Fig. 32

Based on many years of experience in this field and using the above theories, the Borough of Scarborough Works Department has prepared three standard drawings to assist the designer.

These tabulations are shown in figures 33 , 34 , and 35 .

Resumé of design factors

1. Rigid Pipe

(a) Trench Conduit $Wd = Cd\ W\ Bd^2$

Use Curve D for the determination of Cd if no other information is available.

Assume W = 120 p.c.f. if no other information is available.

(b) Positive Embankment Conduits - $Wc = Cc\ W\ Bc^2$

Assume μ = 0.6 Assume K = 0.33

Assume P =+0.7

Assume Rsd = +0.5

(c) Negative Embankment Conduits - $Wc = Cn\ W\ Bd^2$

Assume μ = 0.2 Assume K = 0.15

Assume Rsd = -0.3 p = 1.0 (usually)

(d) Induced Trench Conduit - $Wc = Cn\ W\ Bd^2$

Assume Rsd = -1.0

Assume K = 0.33

(e) Tunnel Installation

$W_t = C_t W\ B_t^2 - 2c C_t B_t$

Assume c = 100 p.s.f. if no other information is known.

Assume L_f = 3.0

(f) Factor of Safety

Assume F.S. = 1.0 for reinforced pipe

F.S. = 1.5 for unreinforced pipe

2. Flexible Pipe

(a) Ring Compression $C - \frac{PS}{2}$

(b) Seam Strength = F.S. x C where F.S. = 4 (assumed)

(c) Buckling Strength = $\frac{C}{12A} \not> \frac{Fb}{FS}$ (ultimate) where

F.S. = 2 (assumed)

(d) Handling and Installation

$$FF = \frac{D^2}{EI} \not> 0.0433 \text{ for small pipe}$$

(e) Deflection.

$$\Delta x = \frac{D\ K\ Wc\ R^3}{EI + 0.061\ E^1 R^3}$$

$E = 30 \times 10^6$

E^1= 700 p.s.i. for good compaction

E^1 = 1400 p.s.i. for excellent compaction

K = 0.44 for good compaction

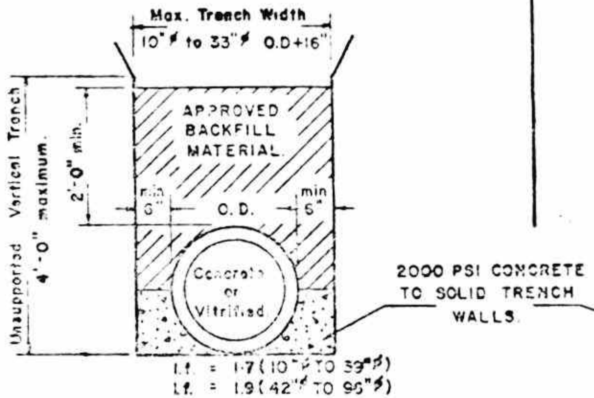
K = 0.22 for excellent backfill

Δx allowable = 5% of vertical dimension.

(f) Ratio of inherent strength to soil resistance

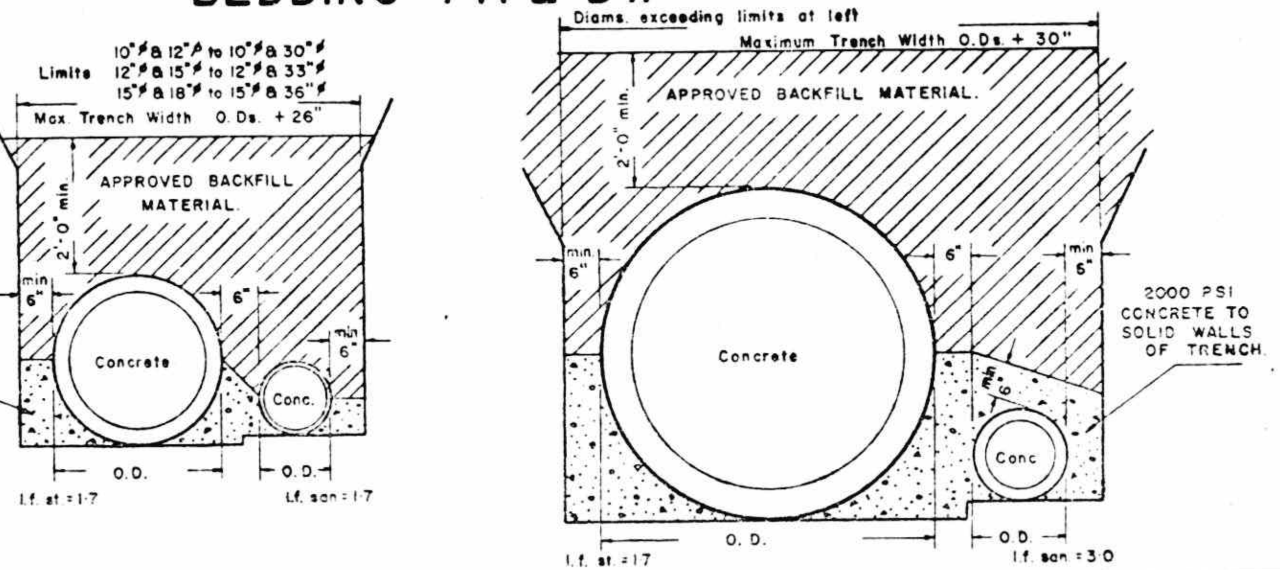
= 10 - 15%

BEDDING TYPE S1.



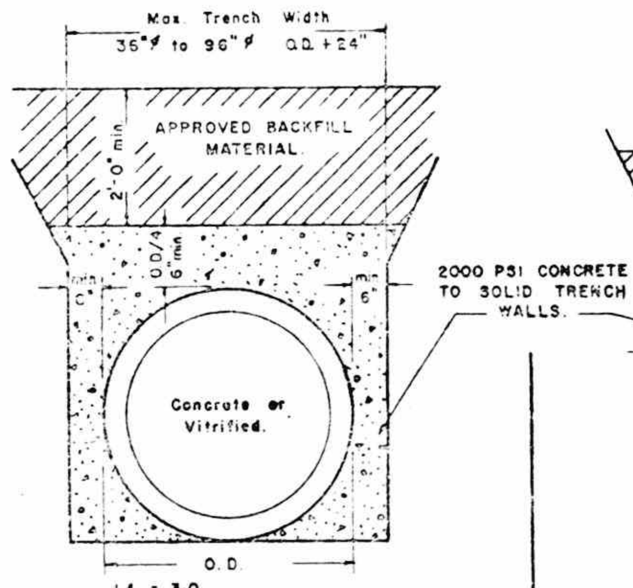
THE APPROVED BACKFILL TO BE COMPACTED IN 6" LAYERS TO 95% STANDARD PROCTOR DENSITY.

BEDDING TYPE D1.

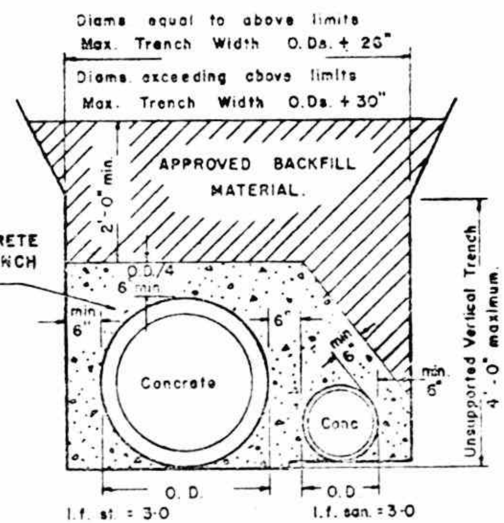


DIA	O.D./4	DIA.	O.D./4	DIA.	O.D./4
10"	4"	30"	9"	60"	18"
12"	4"	33"	10"	66"	20"
15"	5"	36"	11"	72"	21½"
18"	6"	39"	12"	78"	23½"
21"	6½"	42"	13"	84"	25"
24"	7½"	48"	14½"	90"	27"
27"	8½"	54"	16"	96"	28½"

FIG. 33



BEDDING TYPE S2.



BEDDING TYPE D2.

SCARBOROUGH
WORKS DEPARTMENT

BEDDING DETAILS
CONCRETE OR VITRIFIED PIPE

DRAWN BY E.F.K.	DIRECTOR OF ENGINEERING
CHECKED BY R.M.A.	W.A. Wright
DATE JAN. 1972	TECHNICAL SERVICES ENGINEER
DO NOT SCALE.	FILE No D-42-1.

SEWERS IN SAME VERTICAL TRENCH

STORM PIPE			10"SAN.		12"SAN.		15" SAN	
DIAM	SPEC	CL	D1	D2	D1	D2	D1	D2
12"	C-14-68	ES	11•	16•				
	C-76-68	IV	14•	21•				
		V	21•	36				
15"	C-14-68	ES	12•	16•	12•	16•		
	C-76-68	IV	15•	22•	15•	22•		
		V	22•	*	22^{IV}•	*		
18"	C-76-68	III	12•	16•	12•	16•	12•	16•
		IV	16•	23•	16•	23•	16•	23•
		V	22•	*	22^{IV}•	*	22•	*
21"	C-76-68	III	10•	16•	10•	16•	10•	16•
		IV	14•	28	14•	23•	14•	23•
		V	23	*	20^{IV}	*	20^{IV}	*
24"	C-76-68	III	11•	17•	11•	17•	11•	17•
		IV	15•	31	15•	27	15•	24
		V	23	*	21^{IV}	*	21^{IV}	*
27"	C-76-68	II	9•	14•	9•	14•	9•	14•
		III	11•	18	11•	17•	11•	17•
		IV	15•	34	15•	30	15•	27
		V	23	*	23^{IV}	*	21^{IV}•	*
30"	C-76-68	II	10•	14•	10•	14•	10•	14•
		III	12•	20	12•	18	12•	18•
		IV	16	*	16•	32	16•	29
		V	23	*	25^{IV}	*	23^{IV}	*
33"	C-76-68	II	10•	15•	10•	15•	10•	15•
		III	12•	19	12•	20	12•	18•
		IV	16	34	16	35	16•	31
		V	26	*	27^{IV}	*	24^{IV}	*
35"	C-76-68	II	10•	15•	10•	15•	10•	15•
		III	12•	20	12•	19	12•	19
		IV	17	*	16•	32	16•	33
		V	28	*	25	*	26^{IV}	*

STORM PIPE			10"SAN.		12"SAN		15"SAN	
DIAM	SPEC	CL	D1	D2	D1	D2	D1	D2
39"	C-76-68	II	11•	15•	11•	15•	11•	15•
		III	13•	22	13•	21	13•	19•
		IV	18	*	17•	34	17•	31
		V	29	*	27	*	25	*
42"	C-76-68	II	11•	16	11•	16•	11•	16•
		III	14•	22	14•	21	14•	20•
		IV	19	39	18	35	17•	33
		V	30	*	28	*	26	*
48"	C-76-68	II	12•	18	12•	17	12•	16•
		III	14•	24	14•	23	14•	22
		IV	20	*	19	*	18•	*
		V	32	*	30	*	28	*
54"	C-76-68	II	13•	19	13•	18	13•	17•
		III	15•	25	15•	24	15•	23
		IV	22	*	20	*	19	*
		V	34	*	32	*	30	*
60"	C-76-68	II	13•	20	13•	19	13•	19
		III	16	27	16•	26	16•	25
		IV	22	*	21	*	21	*
		V	35	*	33	*	32	*

STORM PIPE			10"SAN		12"SAN		15"SAN	
DIAM	SPEC	CL	D1	D2	D1	D2	D1	D2
66"	C-76-68	II	14•	21	14•	20	14•	20
		III	17	28	16•	27	16•	26
		IV	23	*	22	*	22	*
		V	*	*	*	*	33	*
72"	C-76-68	II	15•	22	15•	21	15•	21
		III	17	29	17	28	17•	28
		IV	24	*	24	*	23	*
		V	*	*	*	*	*	*
78"	C-76-68	II	16•	22	16•	22	16•	21
		III	18	30	18•	29	18•	28
		IV	25	*	24	*	24	X
		V	*	*	*	*	*	*
84"	C-76-68	II	16•	23	16•	23	16•	22
		III	19	31	19•	31	19•	29
		IV	26	*	25	*	25	*
		V	*	*	*	*	*	*

NOTE: ALL CONCRETE SANITARY PIPE TO BE C-14-68 EXTRA STRENGTH EXCEPT WHERE IV IS SHOWN IN WHICH CASE C-76-68 CL IV REINFORCED CONCRETE PIPE IS TO BE USED.

ALL ASBESTOS CEMENT SANITARY PIPE TO BE C-428-67 CLASS 2400

USE 15" CONCRETE MAXIMUM DEPTHS FOR 14" ASBESTOS CEMENT PIPE.

• INDICATES STORM PIPE IN EMBANKMENT CONDITION.

* INDICATES MAXIMUM DEPTH TO INVERT OF AT LEAST 35'

NOTE: TABLES TO BE USED TO 35' ONLY.

SCARBOROUGH
WORKS DEPARTMENT

MAXIMUM DEPTH TO INVERT
CONCRETE STORM PIPE 12"ø TO 84"ø
(CONCRETE OR ASBESTOS CEMENT SANITARY)
IN SAME VERTICAL TRENCH

DRAWN BY F.H. WILLIAMS
CALCULATED BY
CHECKED BY R.M.A.
DATE JAN. 1972

DIRECTOR OF ENGINEERING

W. A. Wright
TECHNICAL SERVICES ENGINEER

DWG Nº D-42-2

FIG. 34

SEPARATE VERTICAL TRENCH

CONCRETE PIPE

PIPE DIAM	ASTM SPEC	CL	BEDDING S1	BEDDING S2
10"	C-14-68	ES	23	*
12"	C-14-68	ES	18	*
	C-76-68	IV	34	*
		V	*	*
15"	C-14-68	ES	20	*
	C-76-68	IV	*	*
		V	*	*
18"	C-76-68	III	15	*
		IV	*	*
		V	*	*
21"	C-76-68	III	15	*
		IV	*	*
		V	*	*
24"	C-76-68	III	16	*
		IV	*	*
		V	*	*
27"	C-76-68	II	12	27
		III	17	*
		IV	*	*
		V	*	*
30"	C-76-68	II	12	27
		III	17	*
		IV	*	*
		V	*	*
33"	C-76-68	II	13	27
		III	18	*
		IV	*	*
		V	*	*

PIPE DIAM	ASTM SPEC	CL	BEDDING S1	BEDDING S2
36"	C-76-68	II	11	21
		III	15	34
		IV	25	*
		V	*	*
39"	C-76-68	II	12	21
		III	16	34
		IV	25	*
		V	*	*
42"	C-76-68	II	14	22
		III	19	35
		IV	32	*
		V	*	*
48"	C-76-68	II	15	23
		III	20	35
		IV	32	*
		V	*	*
54"	C-76-68	II	16	24
		III	20	*
		IV	33	*
		V	*	*
60"	C-76-68	II	16	25
		III	21	*
		IV	33	*
		V	*	*
66"	C-76-68	II	17	25
		III	23	*
		IV	34	*
		V	*	*

PIPE DIAM	ASTM SPEC	CL	BEDDING S1	BEDDING S2
72"	C-76-68	II	18	26
		III	23	*
		IV	34	*
		V	*	*
78"	C-76-68	II	18	27
		III	23	*
		IV	35	*
		V	*	*
84"	C-76-68	II	19	27
		III	24	*
		IV	35	*
		V	*	*
90"	C-76-68	II	20	28
		III	24	*
		IV	35	*
		V	*	*
96"	C-76-68	II	20	28
		III	25	*
		IV	35	*
		V	*	*

TABLES TO BE USED TO 35' ONLY

* INDICATES MAXIMUM DEPTH TO INVERT OF AT LEAST 35'.

SCARBOROUGH
WORKS DEPARTMENT

MAXIMUM DEPTH TO INVERT
CONCRETE PIPE 10"ø TO 96"ø
IN SEPARATE, VERTICAL TRENCH

DRAWN BY F.H. WILLIAMS	DIRECTOR OF ENGINEERING
CALCULATED BY E.J.K.	TECHNICAL SERVICES ENGINEER
CHECKED BY R.M.A.	
DATE JAN. 1972	DWG. Nº D-42-3

FIG. 35

SEPARATE VEE TRENCH

PIPE DIAM	CL	BEDDING S1	BEDDING S2
42"	II	13	21
	III	18	31
	IV	29	*
	V	*	*
48"	II	13	19
	III	17	28
	IV	26	*
	V	*	*
54"	II	14·	19
	III	17	26
	IV	25	*
	V	*	*
60"	II	15·	19
	III	17·	25
	IV	24	*
	V	*	*
66"	II	16·	19·
	III	18·	25
	IV	24	*
	V	36	*
72"	II	16·	20·
	III	19·	24
	IV	28	*
	V	35	*
78"	II	17·	20·
	III	19·	24·
	IV	24·	*
	V	34	*
84"	II	18·	21·
	III	20·	25·
	IV	25·	*
	V	34	*
90"	II	18·	22·
	III	21·	26·
	IV	26·	*
	V	34	*
96"	II	19·	23·
	III	22·	27·
	IV	26·	34
	V	34·	*

SEWERS IN SAME VEE TRENCH

STORM PIPE			10" SAN.		12" SAN.		15" SAN.	
DIAM.	SPEC.	CL	D1	D2	D1	D2	D1	D2
42"	C-76-68	II	11·	16·	11·	16·	11·	16·
		III	14·	21	14·	20	14·	20·
		IV	18	*	17·	33	17·	31
		V	28	*	27	*	25	*
48"	C-76-68	II	12·	16·	12·	16·	12·	16·
		III	14·	22	14·	21·	14·	21·
		IV	18·	34	18·	32	18·	30
		V	27	*	26	*	25	*
54"	C-76-68	II	13·	17·	13·	17·	13·	17·
		III	15·	21·	15·	21·	15·	21·
		IV	19·	33	19·	31	19·	30
		V	27	*	26	*	26·	*
60"	C-76-68	II	13·	18·	13·	18·	13·	18·
		III	16·	22·	16·	22·	16·	22·
		IV	20·	31	20·	31	20·	29
		V	27	*	26·	*	26·	*
66"	C-76-68	II	14·	19·	14·	19·	14·	19·
		III	16·	23·	16·	23·	16·	23·
		IV	20·	31	20·	31	20·	30·
		V	27·	*	27·	*	27·	*
72"	C-76-68	II	15·	20·	15·	20·	15·	20·
		III	17·	24·	17·	24·	17·	24·
		IV	21·	31	21·	31·	21·	31·
		V	28·	*	28·	*	28·	*
78"	C-76-68	II	16·	21·	16·	21·	16·	21·
		III	18·	24·	18·	24·	18·	24·
		IV	22·	31·	22·	31·	22·	31·
		V	28·	*	28·	*	28·	*
84"	C-76-68	II	16·	22·	16·	22·	16·	22·
		III	19·	25·	19·	25·	19·	25·
		IV	23·	33·	23·	33·	23·	33·
		V	29·	*	29·	*	29·	*

NOTES:

1. ALL CONCRETE SANITARY PIPE TO BE C-14-68 EXTRA STRENGTH.
2. ALL ASBESTOS CEMENT SANITARY PIPE TO BE C-428-67 CLASS 2400
3. VEE TRENCHING MAXIMUM DEPTHS HAVE BEEN CALCULATED FOR TRENCH SLOPES OF 1:2 OR STEEPER.
4. USE 15" CONCRETE MAXIMUM DEPTHS FOR 14" ASBESTOS CEMENT PIPE.

· INDICATES PIPE IN SEPARATE TRENCH OR STORM PIPE IN SAME TRENCH IN EMBANKMENT CONDITION.

* INDICATES MAXIMUM DEPTH TO INVERT OF AT LEAST 35'.

TABLES TO BE USED TO 35' ONLY.

SCARBOROUGH
WORKS DEPARTMENT

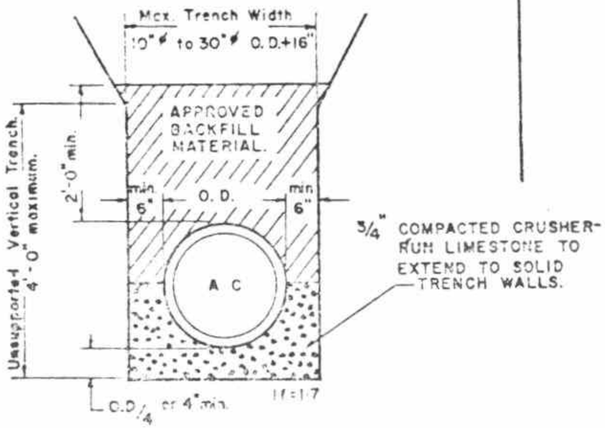
MAXIMUM DEPTH TO INVERT
CONCRETE STORM PIPE 42"Ø TO 96"Ø
IN VEE TRENCH

DRAWN BY F.H. WILLIAMS	DIRECTOR OF ENGINEERING
CALCULATED BY	TECHNICAL SERVICES ENGINEER
CHECKED BY R.M.A.	
DATE JAN. 1972	DWG. Nº D-42-4

FIG. 36

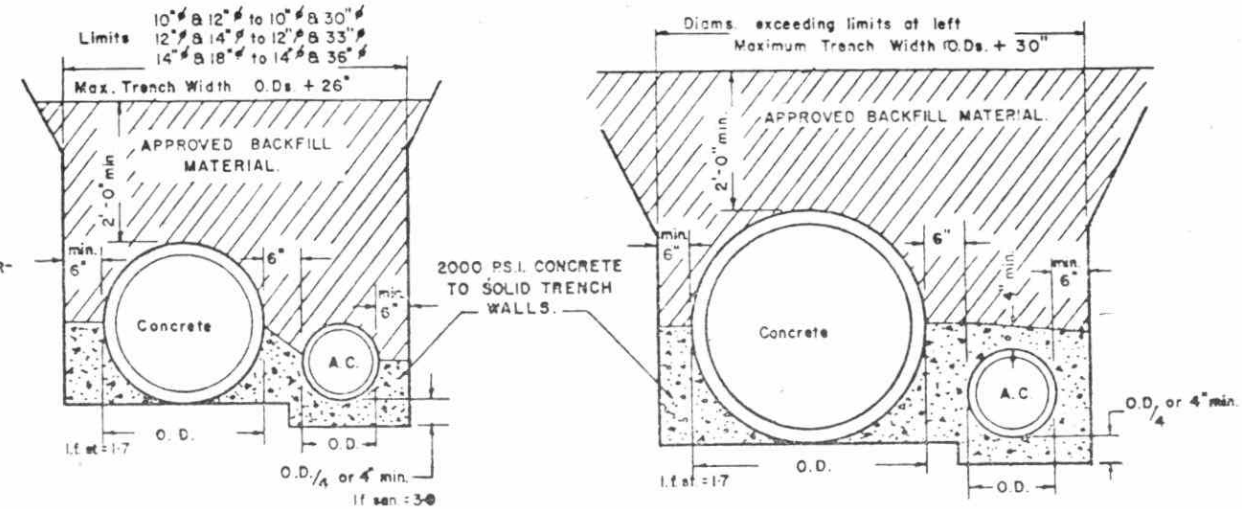
FIG. 37

BEDDING TYPE A1.

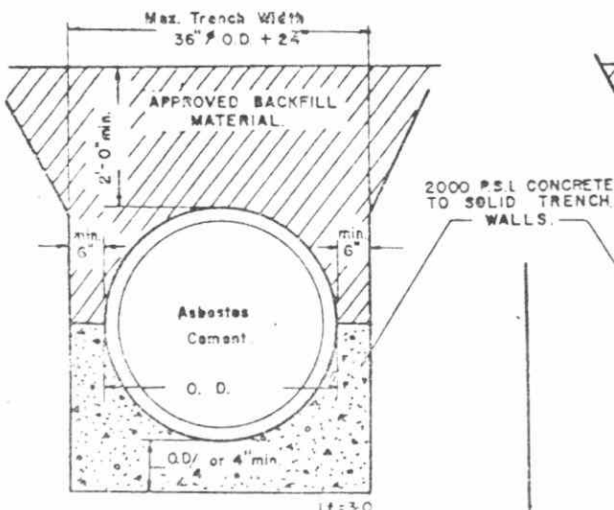


THE APPROVED BACKFILL TO BE COMPACTED IN 6" LAYERS TO 95% STANDARD PROCTOR DENSITY.

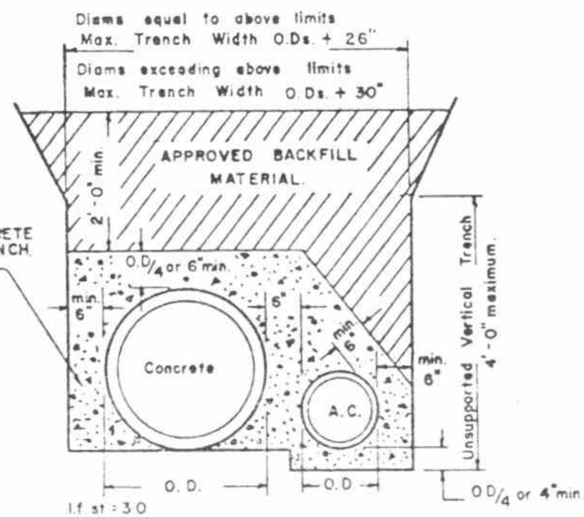
BEDDING TYPE D1.



DIA.	O.D/4	DIA.	O.D/4	DIA.	O.D/4
10"	4"	16"	4½"	24"	6½"
12"	4"	18"	5"	30"	8"
14"	4"	20"	5½"	36"	9½"



BEDDING TYPE A2.



BEDDING TYPE D2.

SCARBOROUGH
WORKS DEPARTMENT

BEDDING DETAILS
ASBESTOS CEMENT PIPE.

DRAWN BY E.F.K.	DIRECTOR OF ENGINEERING
CHECKED BY R.M.A.	TECHNICAL SERVICES [illegible]
DATE JAN. 1972	
DO NOT SCALE	FILE Nº D-4[illegible]-1.

ASBESTOS CEMENT PIPE C-428-67

PIPE DIAM	CLASS	BEDDING A1	BEDDING A2
10"	1500	10	*
	2400	*	*
	3300	*	*
	4000	*	*
	5000	*	*
12"	1500	8	*
	2400	21	*
	3300	*	*
	4000	*	*
	5000	*	*
14"	1500	8	21
	2400	17	*
	3300	*	*
	4000	*	*
	5000	*	*
16"	1500	7	16
	2400	14	*
	3300	33	*
	4000	*	*
	5000	*	*

PIPE DIAM	CLASS	BEDDING A1	BEDDING A2
18"	2400	12	*
	3300	22	*
	4000	*	*
	5000	*	*
20"	2400	11	*
	3300	18	*
	4000	30	*
	5000	*	*
24"	2400	10	22
	3300	14	*
	4000	20	*
	5000	35	*
30"	3300	12	28
	4000	15	*
	5000	21	*
36"	4000	11	20
	5000	13	29

VITRIFIED CLAY PIPE

PIPE DIAM	C.S.A. SPEC	CL	BEDDING S1	BEDDING S2
10"	A-60·1 1969	ES	35	*
12"	A-60·1 1969	ES	25	*
15"	A-60·1 1969	ES	20	*
18"	A-60·1 1969	ES	19	*
21"	A-60·1 1969	ES	19	*
24"	A-60·1 1969	ES	20	*
27"	A-60·1 1969	ES	18	*
30"	A-60·1 1969	ES	18	*
33"	A-60·1 1969	ES	18	*
36"	A-60·1 1969	ES	15	34

* INDICATES MAXIMUM DEPTH TO INVERT OF AT LEAST 35'

TABLES TO BE USED TO 35' ONLY

IN SEPARATE VERTICAL TRENCH

SCARBOROUGH
WORKS DEPARTMENT

MAXIMUM DEPTH TO INVERT
ASBESTOS CEMENT AND VITRIFIED CLAY
IN SEPARATE VERTICAL TRENCH

DRAWN BY F.H. WILLIAMS
CALCULATED BY [illegible]
CHECKED BY R.M.A.
DATE JAN. 1972

DIRECTOR OF ENGINEERING
TECHNICAL SERVICES ENGINEER
DWG Nº D-43-2

FIG. 38

SUMMARY

It is hopeful that this review of the theories of loads and supporting strengths of buried conduits has made one aware of the many factors influencing the selection of a conduit.

Particularly, it should be clearly understood, that the design is only as good as the construction control. Conversely, the designer must be fully aware of the feasibility of his proposals and the limitations of construction practice.

W Elliott.

W. A. Elliott, P. Eng.,
SUBDIVISION CONTROL ENGINEER
Works Department
Borough of Scarborough

ACKNOWLEDGEMENTS AND APPRECIATION

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Mr. R. Emby, P. Eng.	Concrete Pipe Ltd.
Mr. K. Laughlin, P. Eng., Sales Engineer	Armco Canada Ltd.
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Modern Talking Pictures Services

W A Elliott

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SEWER APPURTENANCES

by

Mr. G. L. Wilson, P. Eng.

Sewer Appurtenances

G. L. Wilson.

Although, use of the term "sewer appurtenances" is common, these items might more properly be called "pipe appurtenances" and in this exercise we will consider everything except straight pipe to be a sewer appurtenance. No attempt is made to do other than generally touch the important aspects of each item (hopefully more details will come out in discussion) and the types of appurtenances will range from the very obvious to the more uncommon.

1. Manholes

Manholes are obviously not providing their proper function unless they:

(a) provide convenient access to the sewer for inspection and maintenance.

(b) cause a minimum of flow interference.

(c) are a durable structure.

Obviously, manholes which provide access to very large storm sewers must be designed individually. Otherwise, they are becoming more standardized. Although, manholes are still made of brick or cast-in-place concrete, the precast concrete structure is now more favoured. For nominal sized pipes, the manhole is normally made with a poured concrete base, a barrel made of 48" i.d. concrete cylinders and the top portion a precast truncated cone to bring the top diameter down to 24" and provide one vertical side for steps. Some flexibility is needed in depths and this is normally provided by several courses of hard burned brick.

Fortunately, many engineers are now agreeing on types of castings for

standard frames and covers. Normally, the cover is approximately 160 lb. in weight with two lift holes. It is now becoming common practice to fit these with loose bolts to prevent entry of sticks.
Steps should be wide enough to hold both feet and in high hydrogen sulphide areas should be made of a stainless steel alloy.
The channel and bench have more effect on hydraulics than any other aspect of the manhole and are usually improperly constructed. The channel should be Ushaped and extend at least up to 3/4 the pipe diameter. Benching should be nearly flat, with just enough slope to drain.
Drops across manholes cause turbulence and should be avoided. Even when the pipe size changes through a manhole the channel should be given a steeper slope but no drop unless absolutely unavoidable.
Because the manhole is usually on undisturbed ground and the sewer usually on compacted granular material, there may be a settlement differential. Pipe joints therefore should be placed as close to the outside walls as possible to provide points of flexibility and reduce shear stresses.

2. Drop Manholes

These should be avoided, if possible, but obviously there will be occasions when the resulting depths and costs to provide a continuous slope are just not justified. In these situations a drop structure should be constructed if the inlet is more than 3' higher than the outlet.
Most municipalities construct the drop outside the manhole, encase it in concrete and connect it to the upstream sewer by a T or Y. This

works well but the drop structure is difficult to clean. The Engineer should not overlook the possibility of increasing the diameter of manhole, connecting the drop to the sewer by a T and strapping it to the inside wall of the manhole.

3. Bends

In some situations the use of bends at manholes should be encouraged. This is particularly true in manholes with opposing flows. The radius of a bend should be 3 pipe diameters minimum, if possible, and should present few cleaning problems, particularly if hydraulic flushers are used. The maximum bend which can be accommodated in a manhole is one which will put the tangent points of the outside curves at apposite ends of a manhole diameter.

Bends are useful also in maintaining the sewer location on a curved street where underground space may be at a premium.

An otherwise well designed sewer system can fail to function to capacity and cause back-up problems due only to a failure to recognize fundamental hydraulic principles in the design and construction of manholes. More frequent use of bends to eliminate opposing flows and construction of proper channels and benches will do much to eliminate this problem.

4. Service Connections

These may also be known locally as building sewers, house connections, sewer laterals, etc.and are simply the connection between street sewer and private property.

Sewer Appurtenances, G.L.Wilson

The design, construction and materials used in service connections should be similar in all respects with the main sewer, with certain precautions. Two single connections in the same trench are usually preferrable to a single connection with a Y at property line to serve two properties, particularly, in event of a connection blockage.

Normally, but not always, the Municipality will assume responsibility for connections to the street line, with the portion on private property being the responsibility of the owner. In older sewers, when a connection has been dug up fo r repairs, it is prudent to put an inspection T on the connection at street line. This allows ease of cleaning and also establishes blockage location and thus, responsibility.

If Y or T on main sewer cannot be installed when main sewer is constructed, the connection to main should be made by use of a saddle, not by insertion through pipe wall. Normally, it will be much more safe, economical and convenient to use a drop structure into the main sewer if the sewer is more than 12' - 14' in depth.

Private sewer arrangements (mains and connections) in multiple family residential and commercial plazas can be a problem. Although, the responsibility for maintenance may rest with the owner, approval of original design rests with the engineer. No hard rules can be developed because building arrangements vary considerably. Basically, however, two rules must be applied.

1. All lengths of sewer should terminate either in a manhole or a terminal cleanout structure.

2. If the sewer runs under the floor of a condominium apartment, town house or maisonette, every occupied unit must be accessible to someone at all times in the event of a blockage under a private area. Storm sewer connections to private property are subject to back-up because the storm sewer is designed to be surcharged every two or three years. It is prudent to ensure that these connections do not go into or under a private building if possible but this cannot always be done. Some municipalities require the owner to post a bond saving the Municipality harmless in such cases. This is really a bluff which would not stand up in the courts. The onus of proof of negligence rests with the owner and this alone will determine where the responsibility rests.
There is no question that backwater valves in storm or sanitary sewer connections do provide a measure of protection from sewer back-up if the cause is a consistently overloaded system. However, there are basically two reasons why their use is questionable in sanitary sewer systems. Firstly, their necessity indicates something seriously wrong with the system. Secondly, they tend to give a false sense of security to the owner, who may put a lot of money into his recreation room or warehouse and have it all spoiled because a "flush-a-bye" or paper towel prevented the valve from closing properly.

5. Syphons

Invariably these are inverted syphons or depressed sewers used to cross under canals, railways, streams or other utilities where it is impossible to maintain regular grades. It is customary to build a double-barrelled

syphon for two reasons: The initial flow is usually low and a relatively small pipe must be used to maintain cleansing velocities; the second pipe is available in later years as flow increases, without further construction being necessary; sewage can be diverted to the empty pipe if repairs are necessary.

All bends in syphons should be as long and smooth as possible and larger than normal friction losses should be allowed, particularly, at the inlet weir or structure.

One of the peculiarities of syphons is the tendency to discharge large amounts of air from the upstream manhole and because of the flow turbulence, this is often accompanied by large discharges of hydrogen sulphide. Sealing the manhole only chases the problem upstream. If hydrogen sulphide discharge will cause an odour problem in adjacent residential areas, an air by-pass pipe from the upstream to downstream manhole may be necessary.

8. Catch Basins.

There are probably as many types of storm water inlets as there are city engineers and each engineer can give valid reasons for his preference. The confusion that this causes among newly graduated engineers, contractors and foundries is quite inexcusable.

Obviously, the efficiency and thus the spacing of catch basin grates varies with the rainfall, street grade and crown but it is generally accepted in this part of Ontario that single untrapped, undepressed gutter grates at 150-200 ft. intervals, with double grates at low points, will collect water

to the limit of the sewer's ability to accept it under our present design criteria.

Anyone interested in obtaining results of scientific experiments on catch basins and grates can obtain the Report of the Storm Drainage Committee (1956) from Johns Hopkins University, entitled "The Design of Storm Water Inlets", or "Generalized Hydraulics of Grate Inlets" by J. J. Cassidy, which appeared in the U.S.A. Highway Research Record in 1966.

Cleaning of catch basins in large urban areas is normally done with a vacuum unit mounted on a truck chassis. Officials using these machines, however, can expect union and resident complaints because their high-frequency, high amplitude noise emission has caused discomfort and industrial deafness.

9. Overflow Provisions.

Perhaps, one of the most important items in storm sewer design and maintenance is not really an appurtenance at all. This is the provision for surface overflow when a rainfall occurs of greater intensity than the capacity of the storm sewer can accommodate.

In every drainage area, surface water should be able to reach an outlet, when storm sewers are overloaded, without causing flooding of buildings or damage to property. Ideally, and in most cases, this provision for overflow will be via the street pattern. However, in other cases, particularly where the street pattern cuts across the direction natural drain-

age, the municipality may be obliged to obtain drainage easements across private residential or industrial property and create thereon a depression which will protect the surrounding properties. The only alternative is to design storm sewers for a storm of 100-year intensity.

October, 1971. *G.L.Wilson. P.Eng.*

THE USE OF COMPUTERS IN SEWER DESIGN

The use of computers for sewer design is not particularly widespread in municipal engineering offices. A recent questionaire returned by approximately 60 large municipalities in the U.S.A. showed that only two used computers extensively for this purpose. However, much more use will be made of computers in future for analysis of existing systems, particularly in larger municipalities where redevelopment will change the input into the system. Regardless of the limited current use, however, computers can be both effective and efficient for design purposes.

Types of Programs.

(1) A simple program can be used to replace the commonly used slide rule or nomograph. This merely calculates the required diameter of a single length of pipe when given proper parameters.

(2) A more complex program can be used to analyse an existing system. This will give the capacity of each length of pipe in the system under current condition and will also indicate how each part of the system would be affected if additional flow should be introduced into the system at any point.

(3) Complex programs have been developed by various engineers in Canada and the U.S.A. which can be used to completely design a complex storm or sanitary sewer system and estimate the construction costs.

These programs may be used in two ways:-

(a) In-House Computer

Although quite common in consulting engineering offices, it is quite probable that most computers owned or leased by municipalities on their premises will be business oriented computers under jurisdiction of the municipal Treasurer. This hardware is often not capable of being used for

technical purposes and often is fully occupied for payroll, personnel records, current and capital accounting, etc.

(b) Computer Time Sharing Terminal

Time sharing is simply the simultaneous use of a computer facility by as few as 2 or as many as 50 users who are situated remotely from the computer and who operate independently of each other. The computer is privately owned and its time is rented to the users, thus providing each with a facility much more elaborate and expensive than he could otherwise afford.

Connection to the time shared computer is made by means of a teletype terminal in the users office, which relays signals to and from the computer by a standard telephone connection. This system is very practical for municipal work because of its large scope, ease of use with minimum of training and relatively low cost.

In the following illustrations, no attempt is made to develop the programs but rather to demonstrate the manner in which already developed programs can be used for design purposes.

Program Type No. 1.

In this program the computer is used merely as a tool to replace the sewer slide-rule or nomograph. The problem that is involved is finding the required diameter of a sewer pipe, given the flow of water, the slope of the pipe and its roughness coefficient.

The program is built around Kutters Formula:-

$$\left[\frac{41.66 + \dfrac{0.00281}{\text{slope}} + \dfrac{1.811}{\text{roughness coefficient}}}{1 + \dfrac{\text{roughness coefficient}}{\sqrt{\text{hydralic radius}}}\left(41.66 + \dfrac{0.00281}{\text{slope}}\right)}\right] \times \sqrt{\text{slope} \times \text{hydralic radius}}$$

and the equation

Quantity of flow = Area of pipe x velocity.

By using this program in the time sharing system in a conversational mode, the exchange would look like this: (the underlined characters are those typed by the operator on request by the computer)

```
RUN: PIPE

PIPE                      AUGUST 18TH, 1971.

WHAT IS THE FLOW IN C.F.S.?      16

WHAT IS THE SLOPE IN FT/FT?      0.01

WHAT IS THE ROUGHNESS COEFF?     0.013

THE DIAMETER OF THE PIPE IS 36 INCHES

GOOD BYE
```

Problem solved.

Program Type No. 2

This Program is a little more complicated. It checks the capacity of an existing sewer system and, therefore, requires a lot more data.

Required data:

Description of the existing system, diameter of pipe, length of pipe, roughness coefficient, slope of the pipe.

Also required is a system of numbering pipes so that the total flow is accumulated as the program moves through the system.

The above information would provide the capacity of the existing system. Additional information must be fed into the program at the same time so that the required capacity of the system can be determined and the amount of any inadequacy found if sewage input is changed anywhere in the system. This additional information, in the case of a sanitary sewer system, would be (1) area served by each length of pipe and (2) the population density or the Industrial coefficient for that particular area. From this data the flow into the system at each length of pipe can be determined.

A program such as this could be run on a small time sharing terminal located in the engineering office. However, since these terminals have a printout capacity of only ten characters (letters and numbers) per second, the best method is to run the program on the local terminal but rather than getting the print out at the terminal, save it on file in the computer memory and use the high speed printing facilities of the time sharing company.

Program Type No. 3

The third type of program is much more complex than the previous two. Programs of this nature design a sewer system and provide cost estimates of the construction. Taking care of all the restraints that might occur makes writing a program like this very difficult. By restraints I mean such factors as - amount of cover on tope of a pipe - the presence of conflicting utilities - the maximum and minimum velocities allowed in a pipe. The strength of different types of sewer pipe - different types of soil to be excavated, etc.

Although most municipalities would use it infrequently, the power of the program is quite obvious. The data that must be provided for this program is the same data that would have to be collected if a sewer system was to be designed manually. After all the data are collected, a sewer system could be designed in a couple of days (most of this time would be spent placing the data into a file in the computer's memory) rather than the two or three weeksthat would be spent using manual design methods.

The following is a list of data that are required for a program such as this:-

- pipe coefficient,
- maximum velocity permitted,
- mininim cover,
- minimum drop in drop manhole,
- fixed fall to be allowed at each manhole,
- land use codes (i.e. residential, commercial, industrial, etc.)
  (can be specified as gallons/acre/day or people/acre),
- peaking factor,
- pipe sizes that are available from manufacturers,

- a cost matrix (i.e. average cost per foot, related to size of pipe and depth of pipe),
- minimum slope permitted with each size of pipe,
- cost of manholes,
- cost of drops.

A general layout of the system is required such that manholes and pipes can be numbered and then the following data can be applied to each length of pipe:-

- pipe number,
- downstream manhole number,
- length of pipe,
- ground elevation at downstream end,
- area tributary to the pipe,
- any conflict along the length of pipe (other utilities).

The program will then design a sewer system such that:-

(1) No pipe grade will be selected that is less than minimum slope specified for that pipe size; that will cause a velocity greater than the maximum velocity chosen; that is less than that depth determined by the minimum designated earth cover.

(2) No pipe downstream will have a smaller diameter than the largest upstream pipe entering the manhole.

(3) No pipe will be selected that has a capacity less than that which will carry the peak design flow.

(4) All pipes will be governed by any controlled point of elevation.

(5) The system will always design that pipe which meets the above requirements yet has the lowest installation cost.

The Appendix is an example problem carried out in a conversational mode on a remote terminal connected to time shared computer. (The underlined characters are those typed by the operator).

CONCLUSIONS

Time sharing, using a remote terminal connected to a centralized computer (owned by others) is the best type of computer system to use as far as sewer design for a municipal engineering office is concerned. (Unless a sufficiently sophiscated in-house computer is available at no cost).

An examination of the three programs indicates that although they are quite effective and efficient, their use has certain limitations to a municipal office. Program Type I is fast and very accurate but if a technican desires one solution (e.g. he is calculating the diameter of one pipe only) it is still quicker to use a slide rule. If he has several pipes to solve, use of the program is advantageous. For calculation like this, the best singular solution is use of one of the small electronic calculators that can be programmed to solve particular equations by the insertion of a key card. Purchase of this equipment should not be made, however, unless the engineer is quite certain that his future needs will not require capabilities of more sophisticated hardware.

Program No. 2 is the most useful to any municipal engineering office, provided that the complete sewer system information has been collected and stored in data files. For example, in the case of heavy rain storms and basement floodings, the sewer system can be checked by computer to find which if any pipes are under capacity, what is required to upgrade the system, and whether the back-up was caused by a deficient system or a local problem.

The third program would have use only in the planning of a large subdivision, development of a new townsite, planning a system for an unserviced municipality, design of a long trunk or interceptor or other work of similar scope.

Although business oriented computers can store and retrieve a vast amount of data on an existing sewer system if the information is really necessary, it would be extremely doubtful that many municipalities would be economically prudent to enter into a time sharing agreement for sewer design purposes alone. If, however, the engineer analyses his volume of contract payments, cut and fill calculations, survey traverse calculations, waterworks network analyses and sewer design requirements, the cost of a time-shared computer terminal (which will probably approximate the salary of one technician) may well be justified.

G. L. Wilson, P. Eng.,

August 18th, 1971.

Explanation of the DATA FILES

There are two DATA FILES that are used in this program.

(1) The data file "CONCOS" contains information relating to the constraints of the system and costs of pipes and manholes. This is all information that has to be collected prior to the execution of the program.

Line #100 - the name of the problem.

Line #110 - manning coefficient, maximum velocity (ft./sec.) minimum cover (ft.), minimum backdrop in drop manhole (ft.), fixed fall to be used in the manholes (ft.).

Line #120 - land use codes (can be specified in either IG/ACRE/DAY, IG/CONNECTION/DAY, or as PEOPLE/ACRE).
(20 values have to be specified even if there are 19 zero's).

Line #130 - peaking factor associated with each of the above land use codes.

Line #140 - the number of surface types, the number of nominal diameters, the number of depths in cost matrix.

Line #150 - nominal pipe diameters (inches)

Line #160 - minimum slope associated with each of the above nominal diameters.

Line #170 - depths to be used in cost matrix.

Line #180 - surface material.

Line #190 - #260 - cost matrix.

Line #270 - #340 - one line for each nominal diameter each line - fixed cost of manhole for benching, unit cost of manhole ($/Ft.), unit cost of backdrop ($/Ft.).

(2) The data file "SYSTEM" contains information relating to the actual pipe network.

Line #130 - sewage system identification.

Line #140 - #310 - each line represents a different pipe - multiple manhole number, pipe number, downstream pipe number, overrider for minimum cover (ft.), length of the pipe, ground elevation at downstream end (ft.), surface type number, area (acres) or number of pipes contributing flow to the pipe (all 20 values to be entered).

8. SAMPLE PROBLEM - Program Type No. 3.
(The data used in this sample problem is not necessarily realistic, but does provide an idea of the scope of the program).

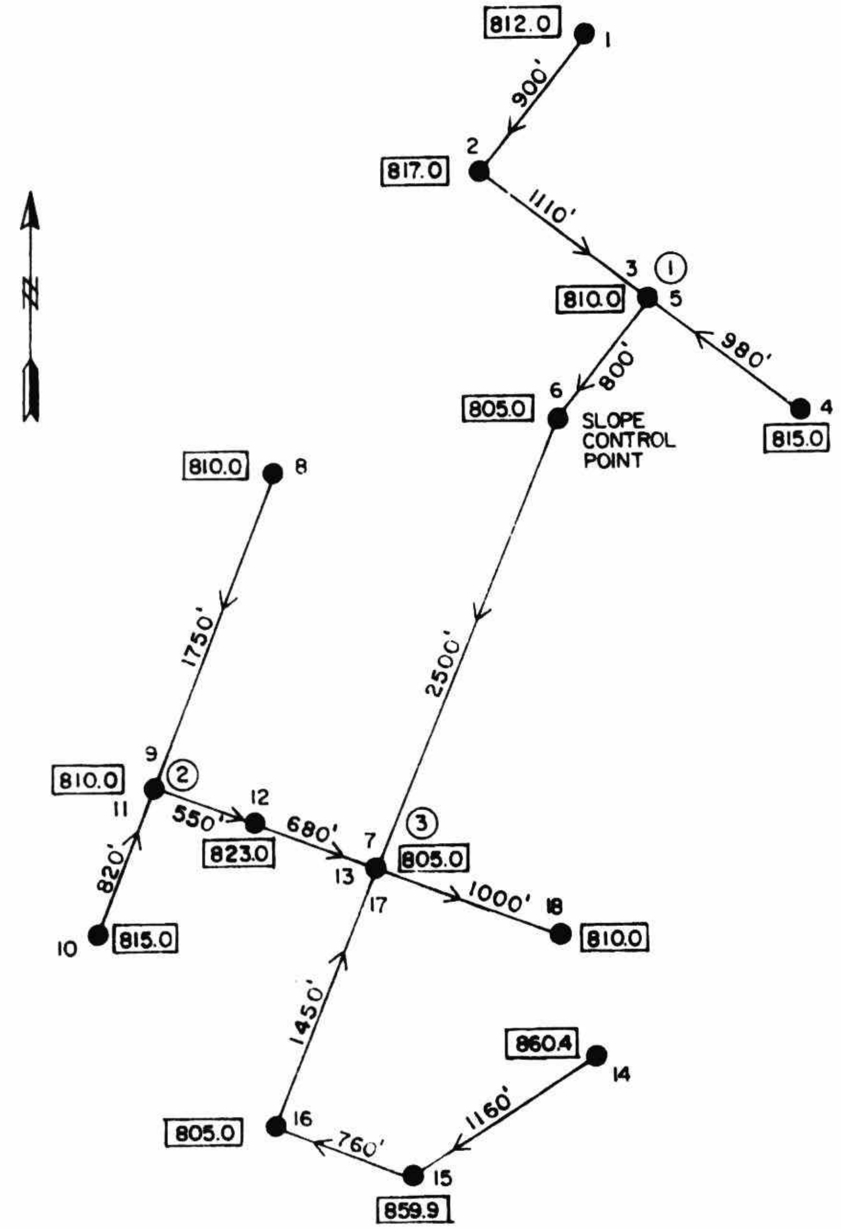
THE DESIGN OF THE SYSTEM SHOWN IN FIG. 1 IS CONTAINED IN THIS SECTION.

'CONCOS' IS THE 'CONSTRAINT AND COST DATA FILE' AND 'SYSTEM' IS THE 'SEWERAGE SYSTEM DATA FILE'.

```
CONCOS     13:53   PC-TOR WED.  28/10/70

100 EXAMPLE PROBLEM
110 .013 8 6 3 .02
120 208 417 458 792 1250 1333 2083 3333 0 1 10*0
130 10*2.4 10*0
140 1 8 6
150 8 10 12 15 18 21 24 27
160 .0034 .0025 .0020 .0015 .0012 .0010 .0008 .0007
170 5 10 15 20 25 30
180 PAVED
190 6.59 11.53 20.79 30.06 39.33 48.59
200 8.85 13.78 23.57 33.36 43.14 52.93
210 11.11 16.04 26.35 36.65 46.95 57.26
220 14.49 19.43 30.51 41.59 52.67 63.75
230 17.88 22.82 34.68 46.53 58.39 70.25
240 21.27 26.20 38.84 51.48 64.11 76.75
250 24.66 29.59 43.00 56.42 69.83 83.25
260 28.04 32.98 47.17 61.36 75.55 89.74
270 200 10 5
280 210 11 6
290 220 12 7
300 230 13 8
310 240 14 9
320 250 15 10
330 260 16 11
340 270 17 12
```

DATA FILE



LEGEND:

PIPE NO.	:	11	PIPE LENGTH	:	900'
MULTIPLE MH NO.	:	②	SCALE	:	1" = 1000'
GROUND ELEVATION	:	810.0			

FIG. 1 SYSTEM LAYOUT

```
SYSTEM     13:55  PC-TØR WED.  28/10/70

130 PRELIMINARY DESIGN
140 0 1 2 0 0 812.0 1 6*0 32.64 13*0
150 0 2 3 0 900 817.0 1 6*0 36.48 13*0
160 1 3 6 0 1110 810.0 1 6*0 44.80 13*0
170 0 4 5 0 0 815.0 1 6*0 48.64 13*0
180 1 5 6 0 980 810.0 1 6*0 39.68 13*0
190 0 6 7 8.5 800 805.0 1 6*0 32.64 13*0
200 3 7 18 0 2500 805.0 1 6*0 101.12 13*0
210 0 8 9 0 0 810.0 1 3*0 8.32 16*0
220 2 9 12 0 1750 810.0 1 3*0 30.08 16*0
230 0 10 11 0 0 815.0 1 3*0 25.60 16*0      DATA
240 2 11 12 0 820 810.0 1 3*0 13.44 16*0     FILE
250 0 12 13 0 550 823.0 1 3*0 8.32 16*0
260 3 13 18 0 680 805.0 1 3*0 10.24 16*0
270 0 14 15 0 0 860.4 1 3*0 11.52 16*0
280 0 15 16 0 1160 859.9 1 3*0 17.28 16*0
290 0 16 17 0 760 805.0 1 3*0 11.52 16*0
300 3 17 18 0 1450 805.0 1 3*0 20.48 16*0
310 0 18 19 0 1000 810.0 1 3*0 8.32 16*0
```

RUNBIG:SANSEW*** (The execution of the program follows)

```
SANSEW       14:01        WED.  28/10/70


ENTER NAME OF CONSTRAINT & COST DATA FILE   ?CONCOS

ARE LAND USE VALUES BY AREA (1), BY CONNECTION (2), OR BY
  PEOPLE PER ACRE (3)  ?1

DO YOU WANT TO DESIGN BY FLUSH OVERT (1) OR BY FLUSH INVERT (2)  ?2

DO YOU WANT TO BYPASS ECHO PRINT-OUT (YES/NO)  ?NO

PROBLEM CONSTRAINTS HAVE BEEN READ.
  DO YOU WANT THEM PRINTED (YES/NO)   ?YES
```

```
EXAMPLE PRØBLEM

  PRØBLEM CØNSTRAINTS

  MANNING CØEFFICIENT = 0.01300
  MAXIMUM VELØCITY    =     8.00 FT/SEC
  MINIMUM CØVER       =     6.00 FT
  MINIMUM DRØP        =     3.00 FT
  FIXED FALL          =     0.02 FT


  LAND USE VALUES (IGD PER ACRE)

               1      2      3      4      5      6      7      8      9     10
              11     12     13     14     15     16     17     18     19     20

             208    417    458    792   1250   1333   2083   3333      0      1
               0      0      0      0      0      0      0      0      0      0
  PEAKING
  FACTØRS  2.400  2.400  2.400  2.400  2.400  2.400  2.400  2.400  2.400  2.400
           0.000  0.000  0.000  0.000  0.000  0.000  0.000  0.000  0.000  0.000




  NØMINAL DIAMETERS & MINIMUM SLØPES HAVE BEEN READ.
    DØ YØU WANT THEM PRINTED (YES/NØ)    ?YES


EXAMPLE PRØBLEM

  NØ. ØF SURFACES  =  1
  NØ. ØF DIAMETERS =  8
  NØ. ØF DEPTHS    =  6

  NØMINAL    MINIMUM
  DIAMETER   SLØPE
      8      0.00340
     10      0.00250
     12      0.00200
     15      0.00150
     18      0.00120
     21      0.00100
     24      0.00080
     27      0.00070
```

DEPTHS & PIPE COSTS HAVE BEEN READ.
DO YOU WANT THEM PRINTED (YES/NO) ?YES

EXAMPLE PROBLEM

SURFACE NO. 1: PAVED

	DEPTH OF INVERT BELOW GROUND (FT.)					
DIAM.	5	10	15	20	25	30
			DOLLARS/FT.			
8	6.59	11.53	20.79	30.06	39.33	48.59
10	8.85	13.78	23.57	33.36	43.14	52.93
12	11.11	16.04	26.35	36.65	46.95	57.26
15	14.49	19.43	30.51	41.59	52.67	63.75
18	17.88	22.82	34.68	46.53	58.39	70.25
21	21.27	26.20	38.84	51.48	64.11	76.75
24	24.66	29.59	43.00	56.42	69.83	83.25
27	28.04	32.98	47.17	61.36	75.55	89.74

MANHOLE COSTS HAVE BEEN READ.
DO YOU WANT THEM PRINTED (YES/NO) ?YES

EXAMPLE PROBLEM

MANHOLE COSTS

NOMINAL DIAMETER	FIXED $	MANHOLE $/FT	DROP $/FT
8	200.00	10.00	5.00
10	210.00	11.00	6.00
12	220.00	12.00	7.00
15	230.00	13.00	8.00
18	240.00	14.00	9.00
21	250.00	15.00	10.00
24	260.00	16.00	11.00
27	270.00	17.00	12.00

```
CØNSTRAINT & CØST DATA FILE (CØNCØS) HAS BEEN READ.
  DØ YØU WANT TØ CØNTINUE WITH THE DESIGN (YES/NØ)    ?YES

DØ YØU WANT RESULTS PRINTED ØN THE TERMINAL (1)
  ØR WRITTEN TØ A FILE (2)    ?1

ENTER NAME ØF SEWERAGE SYSTEM DATA FILE  ?SYSTEM


SEWERAGE SYSTEM ID:
  PRELIMINARY DESIGN


DØ YØU WANT FLØW CØNTRIBUTIØNS PRINTED (YES/NØ)    ?NØ
```

PRELIMINARY DESIGN

MH NØ	SURF TYPE NØ	DIA	PEAK FLØW MIGD	GRØUND ELEV	UPSTRM INVERT ELEV	LNTH FT	SLØPE	DWNSTRM INVERT ELEV	UNIT CØST $/FT	CØST $
1	1	0	0.1632	812.00	805.33	0	0.00000	805.33	0.00	0.00
2	1	8	0.3455	817.00	805.33	900	-.00340	802.27	12.82	11538.20
3	1	10	0.5695	810.00	802.25	1110	-.00250	799.48	18.92	20999.11
4	1	0	0.2432	815.00	808.33	0	0.00000	808.33	0.00	0.00
5	1	8	0.4415	810.00	808.33	980	-.00510	803.33	8.24	8071.93
6	1	12	1.1742	805.00	799.46	800	-.00497	795.48	16.08	12866.37
7	1	15	1.6797	805.00	795.46	2500	-.00233	789.63	24.85	62122.87
8	1	0	0.0158	810.00	803.33	0	0.00000	803.33	0.00	0.00
9	1	8	0.0730	810.00	803.33	1750	-.00340	797.38	11.18	19557.94
10	1	0	0.0487	815.00	808.33	0	0.00000	808.33	0.00	0.00
11	1	8	0.0742	810.00	808.33	820	-.00610	803.33	8.24	6754.07
12	1	8	0.1630	823.00	797.36	550	-.00340	795.49	30.17	16595.88
13	1	8	0.1825	805.00	795.47	680	-.00340	793.16	29.45	20028.12
14	1	0	0.0219	860.40	853.73	0	0.00000	853.73	0.00	0.00
15	1	8	0.0547	859.90	853.73	1160	-.00340	849.79	9.94	11528.08
16	1	8	0.0766	805.00	838.97	760	-.05347	798.33	18.57	14110.46
17	1	8	0.1156	805.00	798.31	1450	-.00340	793.38	10.68	15488.85
18	1	15	1.9936	810.00	789.61	1000	-.00329	786.32	40.53	40534.38

SYSTEM CØST = $ 260196.27

PRELIMINARY DESIGN

THRØUGH MANHØLE RESULTS

NØ.	GRØUND ELEV.	UPSTREAM DIAM.	UPSTREAM INVERT	DØWNSTREAM DIAM.	DØWNSTREAM INVERT
1	812.00	0	0.00	8	805.33
2	817.00	8	802.27	10	802.25
4	815.00	0	0.00	8	808.33
6	805.00	12	795.48	15	795.46
8	810.00	0	0.00	8	803.33
10	815.00	0	0.00	8	808.33
12	823.00	8	795.49	8	795.47
14	860.40	0	0.00	8	853.73
15	859.90	8	849.79	8	838.97
16	805.00	8	798.33	8	798.31
18	810.00	15	786.32	15	786.32

PRELIMINARY DESIGN

NØ.	DRØP FT.	DRØP $/FT.	DRØP $	DEPTH FT.	DEPTH $/FT.	DEPTH $	FIXED $	TØTAL $
1	0.00	0.00	0.00	6.67	10.00	66.67	200.00	266.67
2	0.00	0.00	0.00	14.75	11.00	162.21	210.00	372.21
4	0.00	0.00	0.00	6.67	10.00	66.67	200.00	266.67
6	0.00	0.00	0.00	9.54	13.00	124.02	230.00	354.02
8	0.00	0.00	0.00	6.67	10.00	66.67	200.00	266.67
10	0.00	0.00	0.00	6.67	10.00	66.67	200.00	266.67
12	0.00	0.00	0.00	27.53	10.00	275.27	200.00	475.27
14	0.00	0.00	0.00	6.67	10.00	66.67	200.00	266.67
15	10.82	5.00	54.11	20.93	10.00	209.32	200.00	463.43
16	0.00	0.00	0.00	6.69	10.00	66.87	200.00	266.87
18	0.00	0.00	0.00	23.68	13.00	307.79	230.00	537.79

TØTAL THRØUGH MANHØLE CØST = $ 3802.92

PRELIMINARY DESIGN

MULTIPLE MANHØLE RESULTS

NØ	GRØUND ELEV	UPSTREAM NØ	UPSTREAM DIAM	UPSTREAM INVERT	DØWNSTREAM NØ	DØWNSTREAM DIAM	DØWNSTREAM INVERT
1	810.00						
		3	10	799.48			
		5	8	803.33			
					6	12	799.46
2	810.00						
		9	8	797.38			
		11	8	803.33			
					12	8	797.36
3	805.00						
		7	15	789.63			
		13	8	793.16			
		17	8	793.38			
					18	15	789.61

PRELIMINARY DESIGN

MH NØ	UPSTREAM NØ	UPSTREAM FT	UPSTREAM $/FT	UPSTREAM $	DØWNSTREAM FT	DØWNSTREAM $/FT	DØWNSTREAM $	FIXED $	TØTAL $
1									
	3	0.00	0.00	0.00					
	5	3.88	5.00	19.38					
					10.54	12.00	126.50	220.00	365.88
2									
	9	0.00	0.00	0.00					
	11	5.97	5.00	29.85					
					12.64	10.00	126.37	200.00	356.22
3									
	7	0.00	0.00	0.00					
	13	3.55	5.00	17.76					
	17	3.77	5.00	18.87					
					15.39	13.00	200.08	230.00	466.71

TØTAL MULTIPLE MANHØLE CØST = $ 1188.81

TØTAL CØST = $ 265188.00

END ØF RUN.

HOW MAINTENANCE CAN AFFECT THE DESIGN

by

Mr. H. Lightwood

This paper to be given by Mr. Harvey Lightwood, Borough of Scarborough Sewer Maintenance Manager, at Design Course on Sewers and Watermains.

HOW MAINTENANCE CAN AFFECT THE DESIGN.

Since being asked to prepare a short paper on the subject of design of Sewers and Watermains from the point of view of maintenance, I have thought it might be appropriate to bring to your attention that we are working within a closed system or cycle of water usage. Raw water, sucked from our Great Lakes System, is made potable by filtration and chlorination, delivered to the user who be-fouls the produce in one way or another and returns it to a treatment plant through the so-called sanitary sewers -- although why they are not called unsanitary sewers, we don't know!

This used water, together with the rainfall runoff water is directed back to the system - the former to be treated, and the latter directly where it is again drawn off to repeat the cycle.

I mention this closed system concept here to bring to your attention that the previous view that products were delivered to a consumer is no longer valid thinking; the way to look at it is that the product is delivered to the user, and the used material must be guided back and recycled into the systems.

A system is made up of many parts, and there are recorded many incidents where the operation of a system has been seriously affected by a malfunction or failure of one of its parts which might have been avoided had the design been different, or better, if the designer had put himself in the position of the persons who would be operating and maintaining the subject utility.

A complete water cycle system designed to service an urban society consists of many appurtenances, each of which must perform without detriment to the functioning of that system.

Components of a Storm and Sanitary Sewer System.

The major component here is the conduit or pipes made of various materials through which must pass the sewage and drainage waters; design here is pretty straight forward, and today is assisted greatly by new jointing techniques which have reduced the problems of infiltration and exfiltration, and at the same time give some flexibility.

It is the design and maintenance of appurtenant structures that we will concern ourselves with at this time.

In every sewer system, we will find all or most of the following:

Manholes
Sewer Connections
Pumping Stations
Street catchbasins
Inlet structures
Outfall structures
Open storm channels.

Manholes. These structures are as the name implies, intended mainly to permit entry to the sewer by workmen in order that they may carry out various functions in the maintenance of the system. Often in the past, the designer of the system forgot the sewer function in providing for the workman. This created operational problems. A manhole can severely affect the hydraulic efficiency of a sewer by creating turbulance during periods of peak flow.

Designers have many times carefully calculated the requirements of a sewer, chosen the correct size of conduit commensurate with grade, and then spotted some standard manhole structures along the line of his system, never realizing that he was neutralizing all his careful work by not considering the effect the manholes would have on his calculated discharge figures. Manhole hydraulics have come under special scrutiny in the Borough of Scarborough, and modifications to some existing structure have demonstrated the benefits of careful design.

To maintain design flows in a sewer that is intended to run nearly full, it is folly to have a manhole that is benched only to the spring line; the resulting turbulance destroys the flow characteristics of the pipe. The manhole must be benched to the obvert so that the flow is undisturbed by the presence of the manhole, and at the same time, the purpose for which the manhole was installed must not be forgotten. Special steps and hand holds must be provided for workmen to get into the pipe for inspection and servicing. There are several aspects of manhole design, such as location spacing venting, but sufficient here to remind you to pay attention to these structures and consider their effect on the operation of the system.

Sewer Connections. The whole purpose of having a sewer is so that connections can be made to it from abutting properties, and so again, our flow characteristics can be affected by the manner in which connections are allowed to be made, the number of connections made and the relative size. Wherever possible, factory-made wye junctions should be installed at the time of original construction, and the designer should make every effort to anticipate the needed connections. In excessively deep sewers, it is quite important that risers be installed at the time of construction and brought to a useable depth below finished grade. It is one thing to construct a sewer on a 66' or 86' wide road allowance during the initial development stage, and quite another to attempt a 25' or 30' deep excavation on a busy thoroughfare when it is mandatory to maintain traffic and other services.

In these days of separate storm and sanitary sewers, it is imperative that a method be devised to make sure that private drain layers know the difference between the storm and sanitary connection lines, and are made to realize the importance of connecting properly. In spite of regulations and inspection, there is an alarming amount of storm water getting into the sanitary sewers. No amount of careful design can offset poor control over connection use.

On well inspected public sewer construction, we are quite satisfied that the sewers are adequately tight against infiltration, yet each new sanitary sewer, after private connections are made, has a marked increase in flow rates during and immediately after a rain storm.

Pumping Stations. Pumping Stations are used primarily to permit the development of areas where gravity flow of waste water to a large sanitary sewer is not possible.

In early times, pumping stations were installed somewhat willy-nilly to avoid the task of excavating very deep trunk sewer lines in order to reach low lying areas, and often such pumping units were installed ahead of trunk sewer extensions into an area, in order to expedite serviced lots; such stations were subsequently abandoned.

The designing of a pumping station is one area where designer and operator are often miles apart in fact and theory. It is not my intention, at this time, to go into the myriad of details that make up a sewage pumping station. Sufficient to say that, in view of current legislations regarding pollution of receiving waters, we require built-in safeguards to assure continuous operation, machinery parts and equipment quickly and easily replaced from local manufacturers, together with automatic status alarm equipment to warn of any malfunction. All this and much more is required to operate a fail-safe lift station. We might have stand-by auxiliary pumping equipment in the event of a major power failure, or a major mechanical breakdown. These are the more sophisticated requirements.

I recall earlier pumping stations that did not have any wash up facilities for workmen, no water supply to hose down the wet well walls or the dry well floor, no lights in the wet well which was a dark and forboding chamber, only seen in part with the aid of a drop lamp which, on one occasion I recall, fell into the sewage and blew the main lighting fuse, and the poor operator had to sit astraddle a 6 inch channel iron cross member in the black hole for

nearly an hour, afraid to move, until someone came looking for him with a flashlight.

The designer of a pumping station should use his vision and imagine himself as the operation mechanic, or electrician, actually performing the various acts in the running and maintenance of the station.

Street Catch Basins. Now we are dealing with storm water. One of the main purposes of the storm sewer is to drain away surplus water from the road allowance. To accomplish this, a pair of catch basins on either side of a street are usually installed, spaced at intervals of 250 to 300 feet.

Until a few years ago, it was customary that the designer would merely show that standard catch basins were to be installed at the points indicated. Today, we have ten different types of catch basins. These variations were all brought about by observing problems in the field, and the designer must consider the conditions likely to prevail and choose the type of catch basin that will perform best under a given situation.

To illustrate how maintenance affects design, I recall one of our subdivisions that was built on rolling terrain which resulted in some of the road grade being rising and falling in the order of 3 and 4%. Standard flat top catch basins were installed at standard intervals. Eventually, a fairly high intensity rain storm hit the area, and although the sewer was theoretically capable of carrying the load, flooding occurred; the vertical curves on the street were inundated to a depth of about 18 inches. The overflow from these low points on the street caused washouts through private property located on the low side of the streets.

Investigation brought to light the fact that storm water flowing down the gutters of streets with grades of 3 and 4% was going too fast to get into a standard catch basin top, and most of it accumulated altogether at the vertical curves. There were, of course, standard catch basins at these points, but the concentration of so much run-off was far more than the catch basin could handle,

and so we developed these fairly deep lakes. The resultant head of water flowing through the catch basin lead interfered with the designed velocity of the sewer, and this, of course, made the situation worse. From this, we learned that on all road grades of 3% or over, side inlet type catch bains should be installed at fairly close intervals, in order to capture the run off in medium amounts along the way. These solutions appear obvious in retrospect. However, it would have saved a great deal of time and money to say nothing of the poor public relations caused by the flooding, if the original designer had been aware of the inherent problems in his plan.

Inlet and Outfall Structures. In the development of a storm sewer system, we come to a point where the conduit ends or begins, and water flowing in a ditch or watercourse is required to enter a sewer, or alternately, is discharged from a sewer to a watercourse. Structures have been designed for both purposes, each particularly adapted to do the job intended without creating any problems of their own. I have seen incidents many times where a storm sewer has been terminated at the banks of a stream where a nice looking concrete structure with a head wall and wing walls has been built, only to find that the increased flow from the storm sewer has eroded away the ground in front of the headwall so that it toppled forward into the hole, presenting a sorry looking mess which reflected a lack of foresight on the part of the designer.

On larger sized storm sewers, inlet and outfall structures are required to have a grill or gate, in order to prevent the casual entry of children. The design of such a gate is important. I can illustrate this when I recall one stormy night when the Road Maintenance Section were called out to a badly flooded area to find that a large down-stream storm outlet structure had been blocked almost completely with debris consisting mostly of grass cuttings and other assorted flotsam and jetsam caught up on the grating over the end of the headwall, muddy water was squirting from the top of the gate like a fire hose. One of the men took a thorough soaking and made an effort to unlock the gate to no avail.

Finally, in desperation, they attached a long steel cable to the gate, and hooking the other end to a big truck, literally ripped the gate off its anchor. The water surged out and in a few minutes, the flooding condition upstream was relieved.

This was all because the designer had indicated that a grill be placed over the sewer mouth, and had illustrated this with a drawing showing the bars in a vertical position instead of a horizontal position, which would allow debris floating on the surface of the water to float through the bars without catching.

I will illustrate this more clearly when I show a few slides demonstrating some of the conditions referred to in this paper. There are, of course, other considerations when designing headwalls that will, no doubt, occur to you.

The positioning of the structure itself in relation to the banks of the watercourse is important; placed too far back, we have a channel which must be maintained and usually becomes heavily weeded and prevents the proper maintenance of the surrounding terrain where, for instance, grass cutting equipment must stop and the work done by hand. The height above the receiving waters is also important at time of heavy run-off. The outfall structure could become submerged, creating a backup in the system, and if placed too high, erosion would follow, creating an unsightly condition or undermining the structure itself, as I have mentioned previously.

Open Storm Channels. This type of storm sewer in built up areas is usually a worked over natural watercourse that has been deepened and widened to such an extent that it is no longer physically or economically feasible to cover them over as an underground sewer.

If a natural watercourse is used as a storm water receiver from developed areas without re-designing the watercourse itself to accommodate

the changing conditions, we are going to have a big problem almost immediately due to erosion.

Another incident is brought to mind where, a few years ago, two fairly large residential subdivisions were developed in a watershed drained by a watercourse through a natural ravine. The subdivision agreements made no reference to the developer doing any work in the watercourse into which all storm sewers were shown to discharge. This particular section was approximately 3000 feet in length and had a series of small natural waterfalls that made up a drop of approximately 90' and was on a hard clay base. Calculations showed that a total discharge of storm water from the developed subdivisions was 500 c.f.s., whereas a natural runoff is only a fraction of this.

As you can no doubt see, the increase in flow played havoc with the terrain; instead of a bubbling brook, we had a roaring cascade of running muddy water that literally swept everything before it. This occurred during the first heavy storm in the summer after the completion of the development. Needless to say, it took us some time and a considerable sum of money before we got things back under control. Over a period of four years, we constructed 8 check dams in order to bring the velocity down under 8 fps to stop scouring. The entire area had to be re-worked to accommodate the new set of conditions, and today, is a very different looking watercourse than it was in its natural state.

The design of check dams, watercourse cross sections and other factors in the development of an urban open storm channel are the result of the trial and error methods used in the past.

I have not gone into detail concerning watermain maintenance and design, as my experience in this field is somewhat limited. However, I can point out that working with pressures of from 40 to 50 pounds presents its own type of problems, the main one being watermain breaks which occur at the alarming rate of approximately 300 a year in Scarborough.

The advent of ductile iron pipe in lieu of cast iron used in the past, plus the installation of pressure regulator valves, it is hoped, will reduce this disruptive and costly maintenance problem.

I hope with these few examples of maintenance problems, I have demonstrated that design originates right from the field. It is, of course, obvious that this should be so. However, it is the redesign and reconstruction required due to some bad experience that wastes time and money. This we should try to avoid by having plenty of feed-back information from the maintenance crews to the designer so that he may incorporate operational experiences in his work.

SEWER AND WATERMAIN CONSTRUCTION PRACTICES

by

Mr. R. W. Rodman, P. Eng.

C.E.A. - O.W.R.C. DESIGN COURSE ON SEWERS AND WATERMAINS

SEWER AND WATERMAIN CONSTRUCTION PRACTICES

(R. W. Rodman, P. Eng.)
(City Engineer)
(Niagara Falls)

In considering any discussion on sewer and watermain construction practices, one must take into account the various items which lead to good construction techniques. The construction methods and skills, of course, are intended to produce a superior job, and it is self-evident that inspection is necessary to ensure good construction practices at all times. Proper inspection facilities are necessary on any and all projects, and throughout this paper frequent mention will be made of inspection requirements and practices.

Any discussion of sewer and watermain installations must deal with the inspection, field layout, office and actual construction points of view, to obtain a general understanding of the whole problem and to achieve the desired end result of a well conceived and executed project. The old adage of a chain being only as good as its weakest link, applies to the construction field as well. Unless adequate precautions are taken during the installation of a particular project, many hours of concentrated and excellent work by the designers will be a complete waste of time. It is the finished product that counts, and to achieve a good end result, it is necessary that (1) a proper design and specification for each project be formulated, and (2) proper construction techniques and inspection procedures be followed. This then stresses the importance of the need for good liaison between the designer or office - the layout crews - the inspectors - and the contractor himself.

Generally speaking, the responsibility for a satisfactory contract (assuming it is adequately specified), is vested in the contractor's hands. The inspector or Municipal representative, has certain duties, but it is my opinion that he must be careful not to tell the contractor HOW the finished job should be achieved. There is a definite need to allow the contractors some ingenuity in introducing new construction procedures, but the end result or intent of the specifications must be met. (i.e. Specifications should be general enough to allow flexibility, but specific enough to ensure the desired results). It is important to emphasize that any new procedures suggested by the contractor, should be referred for approval to a higher authority. The higher authority might be the Chief Works Inspector or the Engineer in charge of the particular project. More than one case has been lost when the contractor said "the Inspector told me to do it this way and I was just following instructions". - It can be seen that this lets the contractor "off the hook", and this must be guarded against at all times. Any good contractor knows how to do a job properly in most instances, but with the profit incentive inherent in the contracting field, there is a general tendency to "get away with" as much as possible. Hence, the need for adequate inspection.

One very important rule for both Municipal employees and/or contractors, is to be completely conversant with the specifications, without falling into the trap of becoming too familiar with them. This over familiarity with a contract sometimes lulls one into thinking that each contract will be the same as the last. Each contract is different, and it is important to understand each job requirement. Specifications such as "Information to Tenderers", "Instructions to Bidders", "Special Provisions or Specifications", "Form of Tender" and the like, vary from job to job and must be understood by all concerned. I would stress the importance of never becoming too lackadaisical on the specifications relating to a particular project.

One may ask "Why the need for inspection? " - One answer would be "to provide adequate safeguards, both for the owner and the contractor, on a continuing scientific and professional basis". An inspector helps to achieve proper "sewer and watermain construction practices". As indicated earlier, without him the contractor would be left to his own means to achieve the desired end result. It is very difficult to persuade a contractor to make major repairs to a particular project under any circumstances, and I would submit that proper inspection helps both the "owner" and the contractor. It is important, however, that the inspector limits his duties to those specified, and does not actually direct the contractor in his construction techniques. Suggestions may be given, but it should be emphasized that they are suggestions only and not directions or orders.

Now - What are some of the tools needed by good inspectors? Some of the points that come to mind would be as follows:

1) Good Laboratory Facilities - In the City of Niagara Falls, we are fortunate in having a laboratory in the basement of our City Hall. Our laboratory is quite complete, and each year we try to add certain new facilities to compliment the various inspection procedures carried out in our City. In the Sieve Analysis Room, gradations are checked on all granular materials used in sewer and watermain construction. In addition, we have a Humidity Room for curing concrete cylinders, and monies have been set up in this year's Budget to acquire a Compression Testing Machine for the concrete cylinders. Records are kept of the performance of various contractors who work in the City of Niagara Falls so that each contractor can be assessed on future works being bid upon. In addition, logs are kept of all soils throughout the Municipality when ever a sample can be taken. Needless to say, samples of the various types of parent material are taken from all construction jobs. In addition, members of our Maintenance Department for both sewers and watermains have been instructed to advise the inspection staff of any repairs being made to existing watermains or sewermains, so that samples can be taken of the various soil materials in the Municipality. In this way, future designs can be approached in a more realistic manner, and construction problems can be anticipated before they are met in many instances.

2) An Inspector must keep good notes of all Construction Projects - Our Inspectors achieve this through proper notes in the form of daily diaries, daily work records, "extra" work orders for payment purposes, a log on the number of working days for each project, and a proper pictorial diary of the project achieved through pictures taken before, during and after the job has been completed. All inspection information obtained through

2) Continued....

appropriate chronicling of the various projects, should be reviewed by the design section of any Municipality to ensure proper future designs, based on construction problems and actual costs.

3) Above all - A GOOD SENSE OF HUMOUR - More than one touchy situation developing from a "difference of opinion" has been alleviated through a proper sense of humour and good common sense.

To keep proper control on construction costs,(particularly when the work is being done by Municipal forces) daily job accounting and inspection records are absolutely essential. On contract jobs, records should be kept of all machinery used, soil conditions, weather conditions, and rate of progress for different jobs. This information can then be used to establish good construction techniques for varying types of work, and also to allow better estimating on future jobs involving similar conditions. The City of Niagara Falls' Inspection Department is presently following this procedure, and it is hoped that more accurate estimates of future construction jobs will be achieved, based on actual soil conditions (or probable soil conditions) and anticipated construction techniques.

SAFETY

There are two main acts governing safety procedure on all construction projects. One is known as the Trench Excavators Protection Act and Regulation 559, and the other is known as the Construction Safety Act. These Acts deal mainly with the duties of the inspector, the duties of the contractor, various types of apparel that must be worn, and different procedures and safety devices that must be used and followed. Safety is a vital part in construction techniques or practices, and it is essential that anyone involved in the construction industry be completely familiar with both Acts.

As indicated earlier, an inspector must guard against directing a contractor on the construction techniques or procedures to be followed. If sheeting or shoring is required, certain minimum requirements are indicated in the Trench Excavators Protection Act. It should be emphasized that certain conditions require more than specified in the Act, and any Municipal inspectors should guard against recommending or approving the type of sheeting to be used. It is my opinion that the sheeting of a trench is the responsibility of the contractor, and it is the duty of the Trench Inspector to ensure that adequate sheeting and shoring is placed in the trench. An example of conditions requiring shoring in excess of that required in the Act would be a trench under artesian conditions. In this instance, it is possible that the contractor might be required to use 12 x 12 timbers, whereas, according to the Act, 8 x 8 or 10 x 10 timbers would be sufficient. I would reiterate that when sheeting is required in a trench, the onus is on the Contractor to provide the proper safeguards. In some instances, steel sheeting is needed because of ground conditions.

It is self-evident that accidents must be prevented on all projects, not only from the all-important safety standpoint, but also to ensure good public relations. Some Municipalities have by-laws which specify the types of barriers, barricades, etc., which must be installed for each construction

project, as well as giving a general indication of the extent and type of detour signs that must be provided by the contractors. The City of Niagara Falls has a Traffic Manual which is a requirement on all construction procedures, and this manual specifies the type and size of signs to be used in all detours and other type of barricades to be used in the Municipality. Procedures to be followed by the contractors wishing to do work in the City are not specified in this manual, but some Municipalities have by-laws which cover this type of procedure. I would recommend that by-laws and traffic manuals be considered in any municipality involved in sewermain and watermain installations, to ensure proper conveniences to the public, and also to ensure adequate safety requirements on the various projects.

In the over-all safety picture, it is obvious that safety must begin with the individual. It is essential that a hard hat be worn on all construction projects, and the law now requires that safety footwear also be worn by all personnel engaged in construction, or working adjacent to a construction site. This would apply to field crews working in the general vicinity of any construction site, as well as the inspectors themselves. Of course, the contractor is responsible for ensuring that all his employees wear the proper safety apparel including safety hats and safety footwear. It is my opinion that a Municipality should set a proper example for the contractors to follow, and this would include any engineers who might visit the site for periodic inspections. The law is quite clear that ANYONE visiting a construction site must wear the proper safety apparel. Proper construction techniques and enforcement of these techniques involve two very simple and basic precepts. The first is that a tidy site is a safe site. Boards with nails protruding, broken glass, etc., have no place at a construction site. The second basic safety rule is that unnecessary risks are never taken, no matter how much time or labour might be saved by so doing.

While discussing various safety aspects and procedures to be followed, I would direct your attention to one section of the Trench Excavators Protection Act which indicates that no tool, machinery, timber or other objects shall be placed in or kept adjacent to a trench in a manner that may endanger the safety of the person in the trench. As you will realize, this is a fairly general statement, which covers a multitude of sins. I would emphasize, however, that it is your duty to ensure a proper and safe project, and any job where a man could be working directly below a bucket of a backhoe or dragline for instance or where a piece of equipment was too close to the trench edge, would constitute a threat to the safety of the project, and the job could be shut down. There are certain regulations in the Trench Excavators Protection Act regarding the shape of the trench, and this shape should be maintained wherever possible. As you may know, the contractor is not to construct an underground utility in a trench over six feet, without the use of sheeting, and that depth without sheeting is only allowed in hard or solid soil. He may have only the first four feet vertical and the banks of the remainder of the trench must be sloped at a one to one slope. It is necessary to be alert to changing trench conditions at all times, and this is particularly true in the winter months where frozen soil could change to a weakened condition during thaw periods. Clay in particular is sometimes violently subject to freeze and thaw periods in that it remolds and consequently weakens the fill material.

One other point I would stress from the Trench Excavators Protection Act indicates that no excavated material shall be placed within two feet of the edge of the trench. Of course, some means must be provided for the workers

to gain ingress and egress to and from the trench. The Act specifies that a ladder must be in the trench at all times when men are working in the trench. One final point on safety in trenching operations would relate to the possibility of danger in an open trench when steel sheeting is being used and a rain storm with lightning is in the area. The consequences of lightning striking the steel sheeting while a man is leaning against it in a wet trench are obvious, and fortunately, most contractors and workers have enough sense to realize this. Once again, as part of your general responsibilities, I would suggest that it would be your moral duty if nothing else to remind them of the possible danger if they do not realize the hazards.

PIPE

One of the fundamental principals in knowing good construction practices is to know what type of pipe is generally used.

1) SEWERMAINS - Some of the pipe used in the construction of sewermains would be vitrified clay, concrete, asbestos cement, plastic and corrugated metal pipe. One of the factors that influence municipalities in their selection of one pipe over another is the use to be made of the sewer (i.e. storm or sanitary), and the amount of filtration (i.e. infiltration or exfiltration) to be allowed.

   WATERMAINS - The following are some of the types of watermains now in general use - concrete pressure pipe, steel, asbestos cement, cast iron and plastic. As with the sewermain installations, certain municipalities are influenced in their selection of the type of pipe being used based on past experience, and in some cases, certain areas are restricted in the use of the materials because of the nature of the soil. One such example would be the conductivity of the soil leading to corrosion of various types of pipe. The same could be said for the sewermain of course.

2) JOINTS - Many types of joints are possible for both sewer and watermain pipe. Each should be installed according to the manufacturer's specifications, but only if the pipe is acceptable to the "owner".

   I will discuss sewer joints mainly in this paper, since most of the problems seem to relate to sewer installations. I might add, however, that although sewer installations seem to involve more frequent problems during construction, watermain problems tend to be more serious. (Example - a break or leak in a trunk watermain).

   It is absolutely essential to have good joints in all sewermain installations, and I would emphasize the word ALL sewermain installations and at each joint. The joint of a sewer system could probably be termed the "weak link" of the system, if all other things are considered to be equal (i.e. the more joints there are the more possible trouble spots). The following is an excerpt from the O.W.R.C. Standard Specification covering concrete pipe (for sizes up to and including 36" in diameter) - "sufficient pressure shall be applied in making the joint to assure that the joint is "home" as defined in the standard installation instructions provided by the pipe manufacturer. Sufficient restraint shall be applied to the line to assure that joints, once "home", are held so by tamping fill material under and along side the pipe, or otherwise. At the end of

2) JOINTS - Continued...

a day's work, the last pipe shall be blocked in such a manner as may be required to prevent creep during 'down time' ". The above specification is an indication of the type of "pull up pressure" that is deemed desirable, and I would also indicate that the use of feeler gauges is a good practice to determine if the pipe is "home" and that the proper amount of tolerance has been achieved.

It is noted that asbestos cement pipe comes in longer lengths than most pipes (i.e. 14' to 16' versus the usual 4' to 6'), and it is also noted that the newer plastic pipe materials can also be provided in fairly long lengths. One example of a plastic pipe which was installed in the Huntsville area, involved lengths of pipe in the thousands of feet continual length. As I understand the particular installation, the pipe was floated to the site in the enormous lengths indicated above. Because of the friction factor and the weight of the long length of pipe, difficulty was encountered in pulling the pipe up on to the ground to the construction site. This apparently was resolved by moving the pipe on to the ground after a rain storm, which cut down on the friction factor between the pipe and the ground surface.

Certain tolerances should be achieved at any joint in the construction of sewermains. The horizontal joint left between the bell and spigot of a concrete pipe should not be any more than a 1/4" in small pipe sizes; no more than 1/2" or so on larger pipes such as 24" to 36". On asbestos cement sewer line installations, it is desirable that feeler gauges should be employed to ensure that the gasket is in the proper position. It is also essential to ensure that the joint itself has not been cracked as a result of excessive pressure used in joining the two pipes.

Generally speaking, pipes have rubber gasket joints, and if rubber gasket joints are specified, it is necessary to make certain that the gaskets are of the proper type for the pipe being used. You may recall my earlier comment about the contractor trying to carry out the job in the most "economical" manner possible. I would advise any municipal official involved in the inspection of a sewermain project, to disbelieve any contractor when he tells you he will "mortar" the joints solidly, and that mortaring is better than a gasket. This is not true. Mortar is too porous - and has the tendency to crack if the pipe moves - thus causing leakage problems. Also, during any inspection, the inspector should ensure that there are no "hanging gaskets" or large gaps in the joints.

PROCEDURES FOR A CONSTRUCTION JOB

Each underground construction job is different, but a certain basic routine will usually be applicable. There are certain "before construction", "during construction", "testing", and "after construction" procedures that are generally followed, and I will attempt to describe a few of them at this time.

1) Before Construction - One of the first things that should be done before any contract is started is to make certain that proper arrangements have been made for utility layouts, and it is advisable for all inspectors to question the contractor on this point before a job is started. It is most

1) Before Construction - Continued....

embarrassing, and sometimes fatal if this procedure is not done, if, for instance, an underground hydro cable is encountered. A visit should be made to the site with the contractor to review the ground conditions, and it is very desirable that a pre-contract meeting be held with the Contractor and the Project Engineer. This preliminary meeting will ensure that various items important to the job are discussed and that a thorough understanding of the project is available to all. It is also advisable to have the Contractor's telephone number at the site and at the homes of the owner's representatives as well. These numbers should be given to all concerned with the job, so that emergencies can be reported, if the need arises. It follows, of course, that the Contractor must have certain 'phone numbers - including the home of the Inspector.

All pipes should be checked for any faults when they first arrive at the job site. The Contractor should not be allowed to dump the pipe off the truck, since this procedure would, of course, be inviting chips and cracks in the pipe. Every pipe should be checked, not just samples, and any chipped or cracked pipe should be set aside. One use made of pipes with chipped bells or chipped spigots would be at manholes, but any pipes with chipped bells or spigots should not be used in the main line unless the chip happens to be of a very minor nature. Pipes with improper dimensions, rough pipes or, of course, the chipped pipes should be rejected, and all rejected pipes should be marked (preferably with paint) and removed from the site as quickly as possible. There is always the tendency to use the pipe somewhere along the way if the pipe is not removed at an early date. The type and class of pipe should be stamped on each pipe, and the date of manufacture should also be indicated. The need for the date of manufacture avoids any "green" pipe being delivered to the site (i.e. pipes that may be "too" new). All pipe and other materials should be stored as clear of the trench as possible, and certainly as specified in the pertinent Safety Act. The storage of materials clear of a trench is obviously a good safety precaution which really adds nothing to the contract price. At the same time, this practice could prevent injury in the event of the trench collapsing because of loading or structural deficiencies.

Before any pipe is installed, it is always wise to check the alignment and grade by the batter boards which have been set up for the sewer installation - if this method is being used. String lines and batter boards have become, generally speaking, standard practice for most installations. On occasion, a transit is used to install each sewer line, and more recently more refined methods such as the use of a laser beam have been instituted. It is generally accepted procedure that at least three batter boards (preferably four) be in use for placing each pipe. The Inspector should be supplied with a copy of all grade sheets used for underground installation, and an inspector should be completely familiar with the use of and the method of producing the grade sheets. The inspector should check each pipe installation, during the course of construction, all the while making certain that the contractor is completely aware that it is his responsibility for the proper installation of the underground facility. It must always be realized that grade, size of pipe and material of pipe all combine to give a capacity of a system, and if the grade or alignment is not correct, the capacity is affected. Any inspector of a sewer line

1) Before Construction - Continued.....

should be insistent that the contractor install each pipe to proper grade and line so that no ponding exists, and that at least 3/4 of a circle and preferably a full circle can be seen from one manhole to the next. As with the installation of sewermains, poor grade and alignment on watermains or forcemains could cause air pockets, which once again affects the capacity of a system. Also, if too great a difference in elevation from that specified occurred in the field, conflicts could also occur with other utilities - thus requiring extensive changes to the other utilities or, perhaps, the utility being installed.

2) During Construction

a) Procedures

Where possible, it is desirable to use the parent material for backfill to guard against differential settlement. When working in clay soils, it is best to pulverize the material during excavation, to allow easier compaction at a later date. This could be done by using a "trencher" machine. Basically, any excavation procedure can be used in sandy or silty materials, although frozen particles should not be used in any backfilling operation because of compaction and differential settlement problems. As an example, if a granular material were substituted for a clay material, differential settlement would probably occur in the winter months. This would be caused by a greater expansion of the clay material during freeze periods, as compared to the granular material. If for instance, the sewer were installed in the middle of the road, this could cause a "swale" along the middle of the road which could be quite dangerous when one considers the possibility of ice pockets in the middle of the road. Of course, the problems with differential settlement must be weighed against the problem of not being able to obtain sufficient initial compaction using the parent material. The reverse would then be true with the trench settling continually.

It is customary practice to start at the lower or outlet end of a sewer job and to work upstream. One of the problems of starting at the upper end would be a need for continual pumping of water from the upstream sewer which would be hampering the installation of the next pipe. When working in the usual manner, the only water encountered is that found in the trench itself. One point of particular interest relates to sewer construction which starts at an existing sewer. It is essential that a substantial bulk head be installed at the outlet of the new sewer where it joins with the existing sewer, to prevent premature flow into the existing sewer before the new sewer line has been completely installed. An example of a possible problem relating to an insufficient bulk head would be the case of a sewer being installed for a short distance and a heavy rainfall occurring. The new sewer line could very quickly become surcharged with water from the trench (including mud and silt) and if this were to escape into the existing sewer extensive damages could occur, particularly if the existing sewer were a sanitary sewer only and was not designed for rainfall. One of the problems that could occur would be the surcharging of the sewer or even blocking of the sewer, with sanitary sewage then backing up into finished recreation rooms. To guard against

2) During Construction - Continued.....

this, it is necessary to ensure that sufficient bulk heads are placed between the existing sewer line and the new sewer line, and that the new sewer line is properly blocked at the end of each day's work or before a rainfall occurs. In installing bell and spigot pipe, it is essential that the bell end of the pipe be placed facing upstream, to permit easy installation of the next spigot. If this procedure were not followed, it would be necessary to move the downstream spigot end every time the next upstream pipe were installed.

The usual methods for sewer and watermain installations involve open cut procedures; (a) open cut with punchings - which is merely an open cut procedure with various planks being installed at certain intervals along the trench (supported by jacks or struts) - this procedure is generally used in fairly solid soils; (b) close sheeting which involves the placement of various sizes of sheeting supported by whalers and struts; (c) tunneling; or (d) bore jacking. Bore jacking usually involves the jacking of a liner plate under an existing roadway or railway, and the material is then "bored" out of the liner plate by means of a special machine. In some installations, the "jacked" casing is used as a tunnel liner and the material is actually mined out of the liner plate. The liner plate is of a larger size than the actual utility to be installed, and provides a shield for it. In some cases, a spongy soil could be encountered and the sides of the trench will tend to crack. In instances such as described above, occasionally a large metal box is used and carried along as the sewer is being constructed, or, of course, a timber box could sometimes be used instead. It should be noted that the pipe layers then work inside the box. In all instances, it is most advantageous that the contractor get in and out of any area as quickly as possible, particularly in soft silty soil that has a tendency to crumble and is very subject to heavy moisture conditions.

On watermain installations, the new pipe is usually connected to the existing watermain using a "dry" method or a "live" method. The "dry" method merely involves shutting off a sufficient length of the existing water line, pumping out that section of water line, cutting out a section of the water line to permit the installation of a "T", and the connection of the new line to this section. It is usually desirable that a valve be installed as close to the new "T" as possible, so that the existing line can be recharged and the remainder of the new line installed in a dry condition. Of course, the alternative is the use of the "live" method, and this merely involves the use of a tapping machine to tap into the existing line under pressure. The usual procedure on this method is with a tapping sleeve and valve, which involves the placement of a sleeve over the existing line to reinforce the line, following which a tap is made into the existing line under pressure with a valve connected to the tap. This keeps any water from entering the new alignment, and also allows the existing line to remain in use.

All beddings on new pipes should be well compacted and should be shaped to provide a uniform support across the entire length of the pipe. A gap in the bedding should be left at the bell and then compacted in place after the next pipe has been placed. The most important section of bedding on any pipe is to the spring line, and it is necessary to ensure that uniform support is achieved with no gaps being left. The

2) <u>During Construction</u> - Continued.....

compaction for the balance of the bedding isn't quite as important, although it should be placed by hand and properly tamped in accordance with specifications. The upper half of the bedding is more of a precaution to prevent the pipe from being disturbed when the balance of the backfill operation is carried out above the pipe. I would stress, however, that the specifications should be met to their fullest extent, although greater emphasis would have to be placed to the lower section of the bedding on the various pipe installations.

Before considering any bedding on a pipe, it is necessary, of course, to ensure that the pipe is lowered properly into the trench. Smaller or lighter pipes can be lowered into the trench by means of ropes. Of course, this becomes very difficult as the pipes become larger, and the need for a special sling with the assistance of cranes or backhoes is sometimes necessary on these larger pipes. As indicated earlier, it is, of course, necessary for all workmen to stand clear of any pipes being lowered into the trench for obvious safety reasons.

In the case of vitrified clay pipe, an external gasket joint is sometimes used. When this gasket is used, pipes are usually brought together by hand with a gap of 1/2" being left between the pipe. The joint is then tightened up by external gasket. A more recent development in the use of vitrified clay pipe involves the use of a "Flex-Lox" joint. This is basically a bell type of joint which has been fastened to the plain end of the vitrified clay pipe. The "Flex-Lox" joint makes up for "certain out-of-roundness" in some of the vitrified clay pipe, and it would appear that this type of joint will prove to be very successful for vitrified clay pipe. Smaller asbestos cement pipes and concrete pipes may also be placed by hand, but most sizes require the use of a lever or jack to bring the joints to within working distance of each other. The use of backhoes is sometimes an expedient manner to push pipes together, but this sometimes causes problems with the bells of the pipes being joined, in that they are chipped by the bucket as the pipes are pushed together. As indicated in the standard specification of the O.W.R.C., it is essential to prevent "creep" if at all possible in pipe. A number of situations have arisen in the past wherein the contractor has installed a pipe in accordance with the manufacturer's directions and later found that a number of the joints have come apart. This movement is more than likely to occur in a soft ground condition which causes the pipe to settle and at the same time causes the joint to pull apart. The use of "come-alongs" which is a system of jacks and cables is one of the recommended methods of helping to ensure proper joining of pipes. As each pipe is installed, the cable is extended, and then connected to the bell of the upstream pipe. This pipe is then "jacked" into position. Good construction practice in this type of instance dictates that the end of the sewer should be anchored at the end of each day's working, to prevent movement in the line.

During installations, the batter boards should be checked at least once each day, particularly in the "freeze-thaw" periods of the year. The batter boards and "hubs" (used to establish elevations) have a tendency to rise and fall, which has caused considerable concern on more than one job, since the proposed elevations are, in some cases, drastically affected by the rise and fall of the "hubs".

2) <u>During Construction</u> - Continued.....

It is necessary to ensure that the maximum width of trench (at the top of pipe) is not exceeded, or the design criteria for the pipe could be sufficiently altered to cause a failure of the pipe. As probably explained in earlier papers, the load on the pipe is equal to CWB^2 (B = the width of the trench at the top of the pipe). With C and W being constants for the backfill material, the loading on the pipe varies as the square of the trench width. Assuming a required width of 2 ft, it can be seen that a 4 ft. trench would impose a load four times that designed for originally. It is important, therefore, to keep a watch on the trench width, in accordance with those conditions stipulated in the specifications. When the trench width becomes three times greater than the diameter of the pipe, an embankment condition is said to exist, and the pipe must be designed accordingly. Unless a note to that effect is indicated on the contract drawings, an inspector must assume that the design has been made for trench conditions, and a special watch should be made to ensure that the trench width at the top of the pipe is not exceeded. In cases of doubt, it is wise for the inspector to consult with the Engineer in charge of the particular project to obtain his viewpoints on the matter.

Thrust blocks or cushion blocks are required on watermains and forcemain installations, and it is important that the blocks be properly placed. Usually these blocks are placed at bends, behind hydrants, at "T's", and at dead ends to prevent the joints from blowing apart under pressure.

The backhoe is the most usual type of equipment used for excavations necessary for sewer and watermain installations. There is, however, a limit to the amount of reach when using a backhoe, and different construction procedures require the use of different types of equipment as well. Where sheeting is used, it is usual practice for a crane and clambucket to be used. In some instances a backhoe with a hydraulic control can be used in a sheeting operation. When using a clam in sheeting operations, the use of a system of cables makes the clam a more difficult and slow piece of equipment to use. A slower pace of operation can be anticipated when compared with a backhoe. A drag line is yet another piece of equipment that has its limited applications. Like the clam, it is operated with a system of cables. While the drag line is limited in the depth of excavation possible, its main advantage is its ability in casting material. It should be noted that drag line buckets and clam buckets are interchangeable on cranes. The same cables may be used for both, except that in the case of the drag line an additional horizontal cable is required as well. It can be seen from this that the clam may be used for the excavation, the fill may be placed on the site of the trench, and later the drag line bucket may be inserted on the crane and the bucket used to cast the excavated material to a more convenient position, well away from the trench.

Bulldozers and front-end loaders have many uses in the construction industry. These pieces of equipment are very useful in the underground installation industry, as they may be used to haul pipe, transfer granular material from one place to another, and also used for dumping bedding and backfill material in the trench.

2) <u>During Construction</u> - Continued....

Several methods are used in backfilling, but the easiest way is to push all the fill into the trench using a bulldozer or front-end loader. In an open field type of operation, this would be quite acceptable, providing the pipe is properly bedded, of course, but in built-up areas or other areas requiring a well compacted backfill material, this procedure is definitely not acceptable except where vibratory compacters are allowed.

Manholes may be constructed out of brick, concrete, reinforced concrete or precast concrete. In most manholes, "benching" is required through the manhole as well as the proper channeling of the invert. This is sometimes known as the berm in the manhole, and it is the shaped section of concrete extending across the manhole and connecting the inverts of inlet and outlet pipes. The berm may be shaped either from poured concrete or a pipe may be extended across, cut off at the spring line, and concrete poured in to fill in the space between the concrete pipe and the sides of the manhole. One of the most important points to keep in mind during the construction of any manhole is to ensure that there is a uniform cross section through the manhole to guard against the possibility of turbulence and possible corrosion or erosion of the manhole or sewer pipes in the manhole. In addition to the construction of manholes, lamp holes and/or cleanouts are sometimes constructed, but these are not usually used in most sewer installations. Lamp holes are merely a means of examining the sewer from the lamp hole to the next manhole by means of lamps and mirrors. Cleanouts are a similar installation but are used more for permitting cleaning out of the sewer line.

When manholes are constructed, the problem of trenches that are too wide once again rears its ugly head. I would suggest, in the case of large excavations around manholes, that additional precautions should be taken to provide support for the pipe entering and leaving the manhole. It has been my experience that one of the most numerous instances of pipes breaking or settling is immediately adjacent to a manhole and/or catchbasin. In addition, it is essential that all manholes be constructed according to the drawing dimensions and that any steel is properly placed. One of the points which I must emphasize is that the proper amount of concrete cover must be provided over all steel used in the construction of manholes. If this precaution is not taken, the manhole will "rust" itself, and possible failure could occur. While speaking on manholes and catchbasins, I would point out that these appurtenances must be built to the required surface elevation. In addition, the location of these appurtenances is very important. Many catchbasins in particular, have been constructed at a wrong location and/or grade, producing drainage problems. The final elevation of a catchbasin should be set just prior to establishing the road grade or curb grade, thus ensuring the proper function of the catchbasin. All too often the catchbasins are constructed to the supposed finished grade before the roadway is built, and when the actual curb and/or roadway elevations are set, the catchbasin is found to be too high or out of proper location. The final 6" to 1' of the catchbasin should be left to near the end of the contract.

2) <u>During Construction</u> - Continued......

At the beginning of this paper, I indicated that adequate laboratory facilities should be made available to ensure proper quality control of the job. In keeping with this line of thinking, I would stress that proper concrete testing procedures must be followed during any manhole construction involving concrete, and that sufficient test cylinders should be taken to check the concrete strength. In addition, numerous air entrainment and slump tests should be taken to ensure proper concrete quality. Bedding material, as well as backfill material when specified, should be frequently tested and it is wise to check the source of these materials before work starts. This avoids problems after the work is started. Records must be kept by the inspector of all materials used.

In addition to proper tests on the actual bedding material, it is essential that all bedding be well compacted. The bedding should be shaped to provide uniform support across entire length of the pipe. As explained earlier, a gap in the bedding should be left at the bell when the pipe is installed, and then placed around the bell and compacted after the next pipe is in place. It is important to realize that bedding is an essential part of the pipe design, and if the bedding is not properly done, the results could produce a bedding worse than if none had been specified. In other words, a class "D" bedding or worse could be the result of insufficient compaction. It should also be noted that bedding material should be constructed evenly on both sides of the pipe, particularly to the spring line, to ensure a proper job.

The main idea of backfilling is to properly support the pipe, and to protect it sufficiently before the remainder of the backfill operation is completed. Backfilling in 6" layers is usually specified with mechanical tampers, although sometimes 1' layer compaction with mechanical tampers is specified on "lesser" type roads. The mechanical tamper type of compaction is usually required on existing roadways, and the higher class roadway requires the smaller lifts of backfill material which, of course, ensures better compaction and ultimately less settlement in the future. It should be noted that mechanical tampers should not be used too close to the pipes, since this practice could lead to damage of the pipe itself. In addition, too much tamping is not desirable, since over compaction sometimes produces the effect of loosening the soil particles. Also, the vibrations that are possible through over compacted or tamped soils could cause damage to the pipe. Periodic compaction tests should be taken in conjunction with optimum moisture tests, to guard against over compaction. I would draw your attention specifically to compaction around manholes and catchbasins, since this area seems to suffer in this regard. How many times has a catchbasin been rendered ineffective because of settlement around a structure?

While speaking on compaction, I would, for all intents and purposes, recommend against "jetting" of trenches to obtain compaction. "Jetting" has not been very successful except in granular materials. In silty clay or clay, "bridging" sometimes occurs, and in fine grained soils, uniform settlement is not usually achieved. In addition, "jetting" in the bedding itself could actually undermine the pipe.

An inspector should watch out for rock outcroppings in the bottom of trenches. Pipes laid on a rock outcropping are subjected to a point loading, and breaks inevitably occur, especially in watermains. In

2) During Construction - Continued......

other words, all the stress occurs at one point in the pipe rather than being distributed over the entire length. It is necessary, therefore, that the rock be removed and a proper bedding installed under the pipe in accordance with the specifications of the contract. Occasionally an existing sewer or other underground utility is partially intersected by the proposed sewer facility. If the proposed sewer goes under the existing utility, one method is to chip out the top of the lower facility. This should be done in extreme cases only, and under no circumstances should the BOTTOM of a sewer be chipped out to permit a crossing such as described above. The reason is obvious. In addition, when existing watermains are crossed and undermined because of a new installation, the watermain must be adequately underpinned and supports placed along and under the watermain across the trench and into the existing parent material of the trench walls.

While speaking on general construction rules for underground installations, I would emphasize the need to keep mud and water out of the utility where ever possible. At the end of each day's work, the last length of pipe should be blocked to prevent "creep" in a new installation, and a water tight plug should be inserted at the end of each connection. When starting the job up the next day, the last length of pipe installed should be checked to make sure there has been no change in the elevation. Of course, as indicated earlier, an additional check of the batter boards would also be necessary at the beginning of each day's operation.

b) Problems

There are many problems that can be encountered during a sewer or watermain installation, and the following are but a few of the more general ones to watch for:

i) The soil condition of the bottom of the trench should be carefully examined at all times. If the sub-base is spongy, it may be necessary to excavate to a greater depth to permit the installation of extra granular material in the trench. This is done in an attempt to stabilize the trench bottom, so that a proper bedding for the pipe can be placed and compacted. This is usually achieved through the use of a coarse stone.

ii) During almost every job, a contractor encounters water problems at one time or another that require the use of pumps. These water problems could result from a rain storm or simply from an underground source of water. In any case, on many occasions, it will be necessary for him to pump the water out of the trench. In these cases, it is essential that the contractor have sufficient hosing to ensure that the water is being pumped to an acceptable distance from the trench. Nothing is more discouraging than being swamped by the same water that is being pumped out! This may sound elementary, but you would be surprised at the number of times this has actually happened to seasoned contractors.

There are other instances where the water problems in trenches cannot be gathered at one point to permit the usual pumping operations.

2) During Construction - Continued

b) Problems

ii) The usual procedure in this case involves a series of collection points, done by what is known as a "well-pointing" technique. "Well-points" involve the installation of pipe driven into the ground at intervals, usually ranging from 2 ft. to 5 ft., depending on the nature of the soil. The ends of the pipes are usually perforated to permit the ground water to enter the pipe. After installation, the pipes are usually flushed, following which the pipes are inter-connected by a collection pipe which is then connected to a pump or series of pumps. The system is similar to a "catchment" system, and the pump then directs the water to a convenient ditch or catchbasin nearby. Well-points are used not only to control the flow of water, but also to stabilize the ground. In silty clay soils, well-points are not found to be very effective, but well-pointing has proven to be effective in areas of quick-sand. In the silty clay soil the fine particles have a tendency to clog the pipes before the water can be effectively pumped out. In soils of a fine nature, a granular filter is usually placed around the outside of each well-point. The usual practice is the placement of an outer casing in the first instance, followed by a flushing and placing of granular material in the casing. The well-point is then inserted inside this outer casing. This method has been found most effective, although the expense of this type of operation can be readily appreciated.

iii) While boring or bore-jacking, one can encounter great problems on sewer installations particularly. These are caused by the casing being driven through differing types of soil. For instance, a clay material could be encountered at the beginning of the bore-jacking operation, followed by a quick-sand operation. In the case of a sewer installation, the casing could then settle through the quick-sand area and the required elevations would be lost. Instances such as described above have occurred in the City of Niagara Falls, and in one instance it was necessary to drive a 42" diameter steel casing to ensure that sufficient flexibility would be allowed to permit the installation of a 12" diameter sewer. A rather costly operation, but at times necessary!

vi) Still another instance of undermining that sometimes occurs is when the contractor is changing from a straight open excavation type of operation to a sheeting operation, or conversely, changing from a sheeting operation to a straight open cut operation. On occasion, a different soil pressure is present, which results in an undermining action. When this type of situation is encountered, I would advise that the elevation of the adjoining pipe should be checked to ensure that it has not changed, since this possibility definitely does exist.

3) Testing

During the course of this paper, I have discussed certain tests or checks that should be made at various stages of construction. At this time,

3) Testing - Continued.....

I feel mention should also be made of the following:

i) Watermain Tests - All watermains should be tested to assure the ability to withstand the design pressure under field conditions. This is done by isolating sections of new watermain, subjecting the main to the given pressure as stipulated in the specifications, and holding this pressure for the prescribed time. In addition, all new watermain installations must be chlorinated. Each length of pipe must have a proper amount of calcium hypochlorine as stipulated in the specifications, and all valves and accessories should be operated during the chlorination process. After a sufficient time has elapsed, the mains should be flushed at the extremities of the water line until the water is equal chemically and bacteriologically to the permanent supply. It is essential that the line be properly flushed to ensure that the chlorine has been reduced to an acceptable level, since a severe taste or health problem could result from over chlorination. Each hydrant should be inspected to ensure that it is "plumb" or straight, which should be the case if a watermain was installed in a proper manner in the first instance.

ii) Sewermain Tests - Preferably after the sewermain has been backfilled, filtration tests (infiltration and exfiltration) should be carried out on sewers to determine if the installation complies with the specifications. Infiltration tests should definitely be carried out if the sewer line is installed below the water table. If a sewer line were installed in a completely dry soil, a more appropriate test would be an exfiltration test on the sewer to see if the sewer loses water. It can readily be seen that an infiltration test on a sewer line that has been installed in a completely dry soil would prove nothing. Of course, if the soil then became saturated at a later date during periods of rain falls, a high infiltration could result which would not be desirable. Hence the need for infiltration and exfiltration tests as the case may require. In addition, I would direct your attention to the use now being made of such devices such as cameras and T.V. apparatus to inspect the interior of sewer lines after they have been installed. The above means of testing sewermains is excellent for detecting internal flaws in the installation such as cracked pipes, hanging gaskets, sewer laterals that protrude into the sewer line, etc. All of these types of flaws would be difficult to detect during the installation of sewer lines, particularly if the sewer is of a small size. In addition, the T.V. and camera inspection can detect water infiltrating into the sewer line and also any sags that may be evident in the sewer line.

iii) Material Testing - The sewer and water pipe material is also subject to periodic testing, and rigid requirements are usually set down in the specifications. Periodic tests, usually by independent testing firms, should be made on these materials. Most specifications require a number of pipes (at the owner's discretion) to be made available for testing at no cost to the owner. This is usually limited to 2% of the material being supplied in most specifications. This

3) Testing - Continued.....

iii) Continued ....

avenue should be followed to ensure proper materials being forwarded for each job. It is much cheaper to reject the pipe before it goes into the ground, both from the owner's point of view and the contractor's point of view.

4) After Construction

One of the most important aspects of sewer and watermain construction is good PUBLIC RELATIONS with the people living in the immediate area. This is, of course, very important during the actual installation, and I would suggest that one means of effecting this is to keep the people advised on progress being made. I would caution, however, against making promises that cannot be met. This is always a danger and must be guarded against. While the contractor may have installed a perfectly acceptable sewer, in that it is completely straight and to grade, most of the people will only recall the restoration that is made to the surface after the sewermain has been backfilled. If the street is left in a muddy condition and restoration is not effected to the road surface for some time, this is the impression that is left with the people abutting this street. This certainly does nothing to improve public relations on the job. It is essential that the contractor be made aware that restoration is necessary immediately after the sewer installation, and it is of paramount importance that this point is followed up by the inspectors. Very often it is the chief engineer or his deputy who receives these complaints directly, and this leaves a bad taste in the mouth both from the administration point of view and the public's point of view. The moral of the story is - anticipate the problem before it happens and make the necessary rectifications.

Good clean-up and restoration procedures are essential, not only for good public relations but also for aesthetic reasons. In my opinion, any restoration to pavement or concrete cuts should be to surfaces that have been properly cut with an appropriate saw in the first instance. This produces the most favourable restoration from an aesthetic point of view, and, in fact, is the recommended practice. In road construction, have you ever seen a new surface of asphalt "butted" against an old ragged surface? I think the answer, generally speaking, would be NO in the case of new asphalt construction, but YES in the case of restoration to sewer installations. The aesthetics of a finished job are being considered much more than in the past, and this progressive trend will certainly go a long way in improving future public relations and in improving the somewhat tarnished image of the Municipal Official.

The above remarks have centred on established methods of construction. In earlier sections of my paper, I indicated that new methods and materials are needed. Two such methods are the so called minor and major badger methods of construction.

At a recent conference, a Mr. David R. Ellis and Dr. Ainsley N. Ede presented a most informative paper in which they suggested that the badger

system promises to be the most significant step forward in pipe laying since the time trenching machinery replaced hand digging. The broad objective of the badger system is to abolish trenching in suitable soils, in favour of forcing a giant blade through the ground introducing the pipe. The basic elements of the system include a knife-like blade, means for mounting the blade in a controllable way, an opticle datum line with reference to which the machine is controlled, equipment for establishing the necessary haulage by traction or cable pull, anchor equipment for resisting the cable pull, equipment for installing semi-rigid pipe such as P.V.C. (Poly Vinyl Chloride), poly ethylene or steel pipe utilizing a lubricant where necessary, alternative equipment for feeding down the semi-rigid and/or flexible pipe, and ancillary systems for pipe connections, telephone ducting, etc. The machines presently being used are for installations at depths of 9 ft. in the larger version, and 5½ ft. in the smaller version. Any soil lifted occasionally by the knife-like blade is noticeable immediately adjacent to the blade, and is quite negligible at lateral distances equal to 20-40% of the operating depths, to each side of the proposed installation.

The two machines are designated as the "major", and the "minor". When accurately graded straight pipes are required, as in the case for sewer installations, the "major" is controlled by reference of a modulated light beam. The transmitter is similar in construction to an opticle level, and defines the line above the ground. The blade of the machine is locked to this line using photoelectric cells fixed to the blade, creating a passage for the pipe at an even distance below the beam and parallel to it. Both grading and steering of the pipe are automatically achieved. A graph record shows the progress of the machine in the grading performance. The methods of introducing pipes are alternatively the draw-in system for semi-rigid pipes, or the bead-down system using a duct immediately behind the blade of appropriate radius according to whether the pipe is semi-rigid or fully flexible.

Fully flexible pipe is used principally by the Badger Minor either in the form of Polythene pressure piping, or in the form of corrugated P.V.C. duct and piping of the type developed for land drainage purposes.

In the aforementioned paper, it was indicated that the Badger Minor machine had been used for work in the United Kingdom, Germany and in the U.S.A. Pipes of installations using the Badger Minor have included - rising main sewers utilizing P.V.C. or poly ethylene, generally in diameters up to 12"; water supply pipe lines using similar materials; telephone ducts using P.V.C. ducts in groups; and telephone and electrical cables which are generally fed down directly off the reels.

It would appear that the Badger equipment can normally operate in any soil conditions short of solid rock at one end of the scale and swamp at the other end. The sheer strength of the soil is the controlling factor, but indications are that the Badger blade can penetrate where ever a hydraulically operated bucket trencher can operate, and under these circumstances, an excellent pipe tunnel is produced.

One of the questions raised on this new method involves the protection of the pipe against corrosion. Anxiety has been expressed about pulling the pipes through the ground and scratching or damaging any protective coating of the pipe. Mr. Ellis, in his paper, indicated that new coatings have been

developed which are virtually unscratchable. In addition, a reduction has been made in the possibility of scratching - since the Mole Expander which is pulled through ahead of the pipe exerts a tremendous side (outward) pressure on the ground as it forms a cylindrical tunnel into which the pipe is laid.

The Badger System, in essence, consists of a large knife-like blade cutting through the earth with a bulk head of sorts at the bottom forming a tunnel. The pipe is then pulled through this tunnel, thereby negating the necessity of an open trench.

The Badger Major is the machine used for greater depths of installation such as 9 ft. The gradients of the pipe installed with the Badger Major (for sewer installations) have varied with a minimum of 1 ft. in 300 ft. The profile of any pipeline installed is recorded automatically on a chart in the cab of the machine, and it has been proven that accuracy of ± 0.5" in 600 ft. is usually obtainable. Horizontal deviation can be greater, but apparently practical experience has shown that in the worst cases, it should not exceed 3". This, of course, is sufficient to permit visual examination of the completed pipeline, and it is usually not sufficient to interfere with the practical operation of the sewer.

Of course, the Badger Minor and Badger Major machines are not without disadvantages. One of the main difficulties in Municipal use would be the interference with other existing underground utilities. However, the machine could be very valuable for installations in areas that do not have existing underground facilities, or areas where exact locations and depths of various utilities are known.

In any case, the system would appear to have great merit, and it would appear that the use of the machines together with either P.V.C. pipe or poly ethylene pipe is being discussed with major companies in Canada at this time. The method will certainly require careful consideration by all utility managements in the near future. As indicated earlier, this could be a major break-through in underground utility installations.

While this paper may have rambled a bit from time to time, what I have attempted to do was to illustrate that many factors go into producing good construction practices for the installation of sewermains and watermains. It is my belief that good construction practices not only include the actual installation procedures, but also involve all the resources of both the contractor and the owner. It is my hope that this paper has emphasized this point, and that you will find yourselves in agreement with this theory.

Thank you.

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RWR*vt